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STRUCTURAL STABILITY EVALUATION SANDY LAKE DAM

by

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Final Report

Approved For Public Release; Distribution Unlimited

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

A stability analysis was conducted for a typical interior dam monolith of the Sandy Lake Dam for the following load cases:

(1) Normal operation.
(2) Normal operation with truck loading (H15-44).
(3) Normal operation with earthquake.
(4) Normal operation with ice.
(5) High-water condition.

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20. ABSTRACT (Continued)

To obtain a detailed stability analysis of the monolith, the supporting characteristics of the foundation material were determined by in situ testing using a pressuremeter to predict the horizontal strength characteristics of the pile-soil system.

Two NX core holes were drilled through typical monoliths to obtain access to the foundation material. The pressuremeter tests were performed and in situ soils data obtained. The horizontal foundation soil modulus was obtained as a variation with depth into the foundation material and with soil deformation for three test positions in each of the two test holes.

A conservative horizontal modulus of subgrade reaction was obtained and used in a three-dimensional direct stiffness analysis to determine the forces and deflections at the top of the foundation piles. A beam on an elastic foundation analysis was performed and the pressure, moment, and deflection along the length of the most critically loaded pile were determined.

The compressive forces, tensile forces, moments, and deflections predicted for the piles for all load cases were acceptable. However, the shear stresses at the top of the piles were predicted greater than the allowable value. Therefore, 40 kips of strut resistance for each of the interior small piers should be provided to correct this deficiency.

The average unconfined compressive strength of the concrete was 5700 psi, which is adequate for this structure. The concrete is well consolidated and uniform in appearance. There is no evidence of alkali-silica or deleterious chemical reaction. The concrete in the interior of the structure appears sound; however, observable deteriorated concrete surfaces should be repaired.

After the deteriorated concrete surface is repaired and strut resistance is provided for pier stability, the useful life of the dam will be appreciably lengthened.

PREFACE

The evaluation of the stability of Sandy Lake Dam was conducted for the U. S. Army Engineer District, St. Paul (NCS), by the Structures Laboratory (SL) of the U. S. Army Engineer Waterways Experiment Station (WES). Authorization for this investigation was given in Intra-Army Order for Reimbursable Service No. NCS-1A-78-75, dated 23 July 1979.

The contract was monitored by personnel of NCS, with principal assistance from Messrs. Jerry Blomker and Roger Ronning. Their cooperation and assistance were greatly appreciated.

The study was performed under the direction of Messrs. Bryant Mather and William Flathau, Chief and Assistant Chief, respectively, SL; and John Scanlon, Chief of the Concrete Technology Division, SL. The structural stability analysis was performed by Dr. Carl Pace and Mr. Roy Campbell. The petrographic examination and core logging were performed under the technical supervision of Mr. Alan D. Buck by Miss Barbara Pavlov and Mr. Sam Wong. The concrete testing was performed by Mr. Mike Lloyd. The computer programming was done by Miss Alberta Wade. Core drilling was under the direction of Mr. Mark Vispi, Geotechnical Laboratory, WES. Dr. Pace prepared the report.

Commanders and Directors of WES during the conduct of the program and preparation and publication of the report were COL John L. Cannon, CE, and COL Nelson P. Conover, CE. Mr. F. R. Brown was Technical Director.

CONTENTS

CONVERSION FACTORS, INCH-POUND TO METRIC (SI) UNITS OF MEASUREMENT

Inch-pound units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
acre-feet	1233.489	cubic metres
feet	0.3048	metres
inches	0.0254	metres
kips (force)	4448.222	newtons
kip · feet	1355.818	newton · metre
miles (U. S. statute)	1609.344	metres
pounds (force) per square inch	6.894757	kilopascals
pounds (force) per inch	175.1268	newtons per metre
square miles (U. S. statute)	2.589988	square kilometres

STRUCTURAL STABILITY EVALUATION
SANDY LAKE DAM

PART I: INTRODUCTION

Background

1. Sandy Lake Reservoir (Figure 1) is one of the six Federal reservoirs located in the headwater region of the Mississippi River, about 120 miles* north of Minneapolis, Minn., and about 70 miles west of Duluth, Minn. Sandy Lake is the easternmost reservoir and is the only one that does not have a common watershed boundary with another headwater reservoir. The dam (Figures 2 and 3) is located on the Sandy River, a little more than a mile above its junction with the Mississippi River and about 250 river miles above Minneapolis. The reservoir controls the runoff from a 421-square mile drainage area and encompasses eight natural lakes, including Sandy, Aitkin, Sandy River, Rat, Flowage, Round, Davis, and Tiesen Lakes.

2. Sandy Lake Reservoir was constructed to store water to improve navigation on the Mississippi River between St. Paul and Lake Pepin. This reservoir was considered to be very valuable because it is much closer to St. Paul than the upper reservoirs of Winnibigoshish, Leech, and Pokegama; it also provides additional reregulation and control of releases from these reservoirs. In addition, for predicted stages of 12.0 ft or more on the Mississippi River at Aitkin, the operation of Sandy and Pokegama Reservoirs is correlated to reduce or prevent flood damages in the Aitkin area. With the canalization of the Mississippi River below Minneapolis, Minn., the demands for storage releases from the reservoir system for navigation have been greatly reduced. Thus, in recent years the reservoirs have been operated primarily for flood

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* A table of factors for converting inch-pound units of measurement to metric (SI) units is presented on page 3.

control, recreation, fish and wildlife conservation, water supply, water quality improvement, and other related uses.

3. The Mississippi River Headwater Reservoir System is one of the oldest projects in the U. S. Army Engineer District, St. Paul. The initial surveys and investigations were begun in 1867, at a time when the country was being opened up for development and settlement. The projects are old and were designed almost completely on the site. Based on the available data, it would appear that the original construction was almost entirely a practical field application in engineering with only basic theory to rely on, with the physical design performed in the field, and with a minimum of documentation. Documentation prior to construction was limited to the amount required to develop the engineering feasibility and requirements for construction, authorization, and funding. Post-construction documentation was generally limited to reporting quantities, costs, and justification for additional work or study.

4. The construction drawings do not give a lot of information, but the information is adequate for evaluating these structures. Little theoretical data are available on the dams in the headwater region, which makes it even more important to evaluate the structures and have ready documentation of their structural condition and to make any rehabilitation or modifications that may be necessary. The structures are old, and an evaluation to determine the extent of repair and rehabilitation is timely.

5. Reservoir data are presented in Table 1 and pertinent dam data in Table 2.

Basin Topography and Geology

6. From its sources in Lake Itasca at about elevation 1448.31,* the Mississippi River descends to about elevation 1338.31 above the dam at Bemidji to about elevation 1298.31 at Winnibigoshish Reservoir and to about elevation 1268.31 at Pokegama Dam, near Grand Rapids. In the vicinity of Aitkin, the river flows at about elevation 1188.31. The region

* All elevations are referred to msl, 1929 adjustment.

through which the river flows is relatively flat and includes numerous lakes and large areas of poorly drained swamplands. The headwaters area is generally covered by a mantle of glacial drift ranging in depth from 100 to 300 ft and is composed of a heterogeneous mixture of sand, gravel, and boulders. Although outcrops of rock are found in isolated areas throughout the area, the most notable area is in the vicinity of Pokegama Dam. Of interest is the effect of the glacial deposits upon the region, particularly in the area lying north of Aitkin where depositions of glacial material created a barrier across the river channel below Pine Knoll, which resulted in the formation of glacial Lake Aitkin. Subsequent erosion cut through the barrier permitting the lake to drain but leaving evidence of the ancient beach ridges surrounding the area and the fertile lacustrine and peat deposits in the old lake bed. Also of economic interest is the presence of merchantable iron ores in the sedimentary rock formations underlying the glacial drift in the area extending eastward from Pokegama Lake and in the Crosby-Ironton area, midway between Aitkin and Brainerd.

Control Structure

7. Sandy Lake Dam was constructed between 1891 and 1895. The control structure and lock was a timber and rock-crib design founded on timber piling. The embankment was an earth fill with timber diaphragm cutoff.

8. The dam was built with a 211.5-ft-long by 30-ft-wide lock chamber and was built to withstand a reversal of head, since the Mississippi River occasionally rises when Sandy Lake is low, causing a reversal of flow in the Sandy River.

9. Extensive deterioration of the superstructure lead to replacement of the timber and rock-crib construction between 1907 and 1916, with a new mass concrete control structure and lock founded on the original timber pile foundation. The basic structure remains in place except for the downstream apron replacement in 1958, conversion of the lock chamber into a stop-log control structure in 1958, and installation

of slide gates in 1971. The embankment sections were raised and lengthened because of high stages experienced during the 1950, 1969, and 1971 floods.

Reconstructions of 1907-1916

10. The original timber superstructure was replaced because the timber deteriorated quickly, continuing maintenance was considerable, and the project became unsafe. Therefore, plans were developed on reconstruction, and the entire superstructure was rebuilt of concrete. The structure had six gated bays 5 ft wide, one log sluiceway 11 ft wide, and a navigation lock 30 ft wide by 163 ft long. One of the gated bays was fitted with stop logs for use as a fishway. The structure was topped by a multiple arch concrete bridge having an 8-ft-wide driving surface. This bridge is presently used for access only. The timber apron was removed and concrete placed on the existing pile caps remaining from the timber crib structure. Gage operating machinery was removed from the old structure and adapted to the new concrete structure. The lock chamber was somewhat different than the original lock, and some new timber piles were required to support the new lock. Also, new timber sheet-pile cutoffs were needed at the new lock. The old reversible bear-trap gate on the lock was replaced by a new 15-ft-high by 32-ft-long I-beam and double-plate steel roller gates that opened by rolling horizontally into slots recessed in the landward lock chamber wall. The gates were hand operated.

Later modifications

11. Several major modifications to the structure have taken place since reconstruction. In 1958, the lock was modified, and five 6-ft-wide by 14-ft-high stop-log gate bays were installed in the lock chamber about 6 ft upstream of the upper lock recess. These stop-log sections were installed to provide additional discharge capacity to the structure. The steel framework for the stop logs consisted of 10 WF 49 uprights anchored into a reinforced concrete base that is cinch anchored into the concrete lock floor. Stop logs are 6- by 6-in. treated timbers. Both lock gates have been permanently stored in an open position in the lock gate recess.

12. Also, in 1958, the downstream timber apron and lock floor were replaced by a 12-in.-thick, reinforced concrete slab on the existing timber piles and pile caps. The downstream portions of the fishway and log sluice piers were removed to facilitate placement of the apron. In addition, a new 10-ft-long SA-23 steel sheet-pile cutoff wall was constructed along the downstream end of the sluice gates and lock chamber, with a portion of the piling (and slab) running parallel to the outside of the lock wall. The existing structures adjacent to the work areas were grouted to assure that any voids under the concrete would be filled. Weepholes, 4-in.-diam black steel pipes, fitted with perforated iron strainer caps, filled with gravel and surrounded by filter gravel, were installed in the new apron floor. A 20-ft-wide section of 12-in.-thick riprap and two 6-in. layers of graded filter material were placed adjacent to the apron. The riprap on the abutments was grouted after placement.

13. In 1971, the original timber gates and operating machinery in the sluice gate were replaced by six new 50-in.-high by 60-in.-wide steel slide gates. The new gates were fitted with 1-3/4-in.-diam stainless steel riser stems and operating benches. The six timber stop-log bays in the lock chambers were also modified. Reinforced curtain walls 12 in. thick were constructed from the lock floor to elevation 1216.81 (11-1/2 ft above the lock floor), and timber stop logs were furnished to elevation 1219.31, giving an overflow barrier with an effective height of 14 ft. New aluminum alloy upstream handrailing and gate walkway gratings were also installed at this time. More detailed information on the Sandy Lake Dam is given in reports by the U. S. Army Engineer District, St. Paul (1973 and 1977).

Objective

14. The objective of the study is to evaluate the stability of the concrete control structure.

Scope

15. The study is limited to a structural stability evaluation of the concrete control structure with consideration given to foundation and concrete properties. The stability analysis was performed in accordance with current Corps of Engineers criteria. To aid in this evaluation, two cores were drilled through the dam and into the foundation. The foundation material was tested in situ in order to determine its supporting capabilities. The cores and foundation material were examined and the structural stability of the dam was evaluated.

PART II: CORING PROGRAM

16. Since Sandy Lake Dam falls into the classification of a low-head dam, limited coring was performed to obtain the properties of the concrete and to obtain access to the foundation material in order to perform in situ testing.

17. Two NX concrete cores were obtained (Part III). The piers are numbered from right to left looking from upstream to downstream. The location of the core holes in piers 1 and 5 are presented in Figure 4. The core holes in piers 1 and 5 were drilled by using a truck-mounted drill rig to core through the roadway and pier.

18. Diamond core bits and 5-ft-long double-tube, swivel head core barrels were used to obtain cores from the concrete. Holes were drilled into the foundation material and a 60-mm pressuremeter probe was inserted to the desired depths to perform the pressuremeter tests. Split spoon tests were performed before the holes were drilled out for pressuremeter testing.

19. The coring program was oriented toward determining:

a. depth of deteriorated concrete.

b. uniformity of concrete with depth.

c. unconfined compressive strength of the concrete, and

d. the foundation material properties by in situ testing, using the core holes as the access to the foundation.

20. The in situ strength of the foundation material is an important factor in the analysis of the stability of the dam, which is supported on timber piles embedded in the foundation material. The drill rig was used to perform pressuremeter tests, standard penetration tests, and to obtain disturbed samples of the foundation material.

21. The coring program was considered a minimum for obtaining representative information on the concrete and foundation material but is adequate for this particular dam. The core holes were not grouted; pipe caps were used to seal the openings in order that the core holes could be used in the future to obtain piezometric data.

22. A representative concrete core and a cut section are shown in Figure 5. The concrete at Sandy Lake Dam was found to be very uniform.

PART III: PETROGRAPHIC EXAMINATION AND CORE LOGS

Samples

23. On 29 October 1979, two NX concrete cores were received at the Structures Laboratory of the U. S. Army Engineer Waterways Experiment Station for tests and examination. The cores were taken from two piers at Sandy Lake Dam, which was constructed in 1909.

24. Both cores were drilled vertically and are identified below:

Core No.	Pier	Elevation	Length
S-P1	1	1223.81 ft	18.6 ft
S-P5	5	1223.81 ft	18.0 ft

Test Procedures

25. The cores were inspected and logged. Pieces from the upper, middle, and lower portions of each core were selected for strength tests. Short pieces from the 10- and 12-ft intervals of S-P1 and from the 12-ft interval of S-P5 were selected for detailed petrographic examination.

26. The piece from the 12-ft depth of core S-P1 was judged to be typical of the concrete in both cores. It was cut along its axis and one sawed surface was then ground smooth for examination with a stereo-microscope.

27. A cement paste concentration was prepared by crushing some of the concrete from core S-P5 and passing it over a No. 100 sieve; this powder was then reground to pass a No. 325 sieve and was examined by X-ray diffraction. The X-ray patterns were made with an X-ray diffractometer using nickel-filtered copper radiation.

Results

28. The nonair-entrained concrete of both cores was similar. Core S-P1 was broken at intervals of 0.2 to 0.5 ft along its entire length. Core S-P5 was broken at intervals of about 0.4 ft below a depth of 4.5 ft. The breaks appeared to be new and were believed to be due to the drilling operation. Since the concrete appeared similar, the greater frequency of breaks in core S-P1 was also ascribed to the drilling operation. The concrete was well consolidated. Maximum aggregate size was judged to be 2 in. in core S-P1 and 1-1/2 in. in core S-P5. The coarse aggregate was composed of granite, granite gneiss, and fine-grained, dark colored, igneous rock particles. The mixed composition and particle shape suggested this was gravel and that some crushing may have been used. The fine aggregate was the same material with more single grains of quartz and feldspar. The logs of these cores are shown in Figures 6 and 7.

29. White material found filling air voids or on the surfaces of some aggregate particles was ettringite. No alkali-silica gel was found to indicate that any alkali-silica reaction had taken place.

30. X-ray diffraction of the cement paste from core S-P5 showed calcium hydroxide, ettringite, tetracalcium aluminate monosulfate-12-hydrate (monosulfoaluminate), tetracalcium aluminate carbonate-11-hydrate and hemicarbonate-12-hydrate (monocarboaluminate and hemicarboaluminate), and probably hydrogarnet, vaterite, and calcite as paste constituents. Aggregate contamination in the same sample was mainly quartz and plagioclase and potassium feldspars, along with smaller amounts of amphibole, mica, and clays. These were reasonable compounds to find in the cement paste and in the aggregate. While calcium silicate hydrate is the most important constituent of the cement paste, its low degree of crystallinity means that it is not usually detectable in X-ray diffraction patterns.

Discussion

31. The concrete from this structure as represented by the cores appeared to be homogenous. The increase in the frequency of breaks in some of the concrete cores is believed to have been due to the small core size and the drilling procedures. Larger diameter cores would probably have improved the condition of the cores. The vertical fracture found at the top of S-P1 is probably related to the cold joint aligned with it.

PART IV: FOUNDATION AND CONCRETE PROPERTIES

In Situ Foundation Testing

32. The foundation supporting characteristics for a piling system based on material properties from samples and laboratory testing in an attempt to obtain in situ material properties is at best approximate. Soil conditions and stress fields can be controlled in the laboratory, but just how faithfully they represent in situ condition is a matter of conjecture. A further complication at Sandy Lake Dam was that the foundation material was gravel and silt, which made the ability to obtain undisturbed samples doubtful.

33. The foundation material was tested in situ by determining the resistance of the soil to horizontal deformation to obtain the structural supporting characteristics of the pile-foundation system. The pressuremeter method was used to measure deformation properties and obtain a rupture or limit resistance of the foundation material.

Pressuremeter Tests

34. In 1933 Kögler (Baguelin, Jiziquel, and Shields 1978) wrote about a pressuremeter for obtaining in situ foundation properties. Since before 1965 the pressuremeter has been used in France for the design of building and bridge foundations. Pressuremeters are now being used in the field in the United States.

35. The pressuremeter probe was placed within a previously drilled borehole at the desired elevation for testing. When necessary, it was housed in a slotted tube that was driven to the desired elevation for testing.

36. Pressure was applied in equal increments and the corresponding volume variations noted at 15, 30, and 60 sec. The data were corrected for calibration, waterhead, etc., and then used in an analysis to obtain the supporting capability for the pile foundation.

Pressuremeter Field Tests and Results

37. Access to the foundation material was obtained by drilling an NX hole through each of two dam piers. A properly sized hole was drilled into the foundation material. The pressuremeter probe was then inserted to the desired elevation, and the test was performed. A pressurized bottle of gas was used as the pressure source for the pressuremeter tests, which were performed at three elevations in hole S-P1 and hole S-P5. The locations of the probe below the bottom of the pier are presented in Table 3 for both core holes.

38. Standard penetration (split spoon) tests were also performed in test holes S-P1 and S-P5, and the results are presented in Table 4.

39. Disturbed samples of the foundation material were obtained for laboratory testing and classification. The foundation material under Sandy Lake Dam was found to be mainly sand, silty sand, and gravel (Figures 8-14). The standard penetration values for this material indicate that it is in general a compact material.

40. The main characteristic of the foundation material, which represents its horizontal supporting capability for a pile foundation, is the subgrade modulus and its variation with pressure and depth into the foundation. The pressuremeter tests gave these data. Plots of data for hole S-P1 are presented in Figures 15-21 and for hole S-P5 in Figures 22-28.

41. The test cavities in hole S-P1 for tests 1 and 3 and for test 2 in hole S-P5 were oversized and the data were given little consideration in the analysis.

42. The recorded pressure must be corrected because of the hydrostatic pressure of water in the tubing and for the probe calibration, which gives the resistance to expansion of the rubber membrane. The corrected pressure curves are presented in Figures 15 and 17 for hole S-P1 and Figures 22 and 24 for hole S-P5.

43. The limit pressures were obtained by plotting pressure VS 1/(volume) and extrapolating the curves to the pressure at 1/(volume) = 0 . The extrapolated pressure determinations are presented in Figures 16 and 23.

44. The shear modulus G depends not only on the slope of the pressure-volume curve but also on the volume of the probe.

45. The average volume is used in calculating the shear modulus as follows:

$$G_M = [535 + \frac{V(I) + V(I + 1)}{2}]\frac{\Delta P}{\Delta \nu}$$

$$= [535 + \frac{V(I) + V(I + 1)}{2}][\frac{P(I + 1) - P(I)}{V(I + 1) - V(I)}] \qquad (1)$$

46. The deformation modulus, which is something roughly equivalent to Young's modulus, is obtained frm the well-known equation:

$$G_M = \frac{E_p}{2(1 + \nu)} \qquad (2)$$

47. Poisson's ratio is used as 0.33, and the resulting deformation modulus is called the Ménard modulus, E_M.

$$E_M = 2(1 + \nu)G_M$$

$$= 2(1 + 0.33)G_M = 2.66G_M$$

The Ménard modulus is presented in Figures 19 and 26 for holes S-P1 and S-P5, respectively.

48. The subgrade modulus k is obtained from the following equations:

$$\frac{1}{k} = \frac{2}{9E_M} \cdot B_o(\frac{B}{B_o} \text{ x } 2.65)^{\alpha} + \frac{\alpha}{6E_M} \cdot B \qquad \text{for } B > 2 \text{ ft} \qquad (3)$$

$$\text{or} \quad \frac{1}{k} = \frac{B}{E_M}(\frac{4(2.65)^{\alpha} + 3\alpha}{18}) \qquad \text{for } B < 2 \text{ ft} \qquad (4)$$

where

B_o = reference pile diameter, 2 ft

B = pile diameter

α = rheological coefficient given in Figures 3-48 of Baguelin, Jiziquel, and Shields (1978).

49. After a representative value of k has been determined, it can be multiplied by the pile diameter to obtain the horizontal modulus of reaction for the support of a pile. The horizontal modulus of reaction of the soil can be used in the piling analysis to obtain deflections, forces, and moments to use in evaluating the adequacy of the pile foundation.

Piling and Concrete Data

50. The 12-in.-diam Norway Pine piling, which supports the monoliths at Sandy Lake Dam, are approximately 15 ft long. The properties of the Norway Pine piling are as follows:

Modulus of elasticity (E) = 1.32×10^6 psi

Shear modulus (G) = 0.45×10^6 psi

Allowable compressive stress parallel to grain = 1100 psi

Allowable tensile stress parallel to grain = 775 psi

Allowable average shear stress = 75 psi

Allowable average shear force per pile = 8.5 kips

Allowable compressive load on a pile = 124 kips

Allowable tensile load on a pile = 0 kips

Allowable moment in a pile = 131,000 in.-lb or 10.9 kip-ft

The properties of Norway wood can be found in many handbooks. One such reference is presented (Southern Pine Association, 1954).

51. The unconfined compressive strengths of the concrete (Table 5) were adequate and since the interior concrete has performed well for over 70 years the structure, with some maintenance, can be expected to perform well for many more years.

52. Since the interior concrete is of good quality, the deteriorated surface concrete should be repaired to ensure that water is not allowed to enter cracks and thus accelerate the deterioration of the interior concrete. There are a number of methods of repair that might be used; but the Upper Mississippi River Headwater Structures are ideal for an economical repair such as:

a. clean surface concrete,

b. fill cracks, and

c. paint on a cementitious coating to rehabilitate the surface concrete.

53. Under some conditions an acrylic-polymer coating of a composition as listed in Tables 6 and 7 might be used. Certain polymers have exhibited good bond and noncracking characteristics when used in ordinary environments. They have also shown good resistance to freezing and thawing environments. The particular polymer to be used should be tested as follows before being used to rehabilitate the surface concrete of the Upper Mississippi Headwater Structures.

a. Determine the resistance of the coating to cracking during extreme temperature changes.

b. Determine the coating's ability to retain good bond capability in freezing and thawing environments.

c. Determine the coating's ability to "breathe" (allow water to escape from the interior concrete through the coating and thus preventing saturation).

PART V: STABILITY ANALYSIS

Conventional Stability Analysis

54. Conventional stability analysis assumes that the base of a structure is rigid and obtains the loads on the piles. This type of analysis does not consider the load distribution due to pile and structure deformations with consideration being given to the strength and supporting characteristics of the soil on the piling system. The monoliths at Sandy Lake Dam are of such size and shape that the assumption of a rigid base is adequate. However, the supporting characteristics of the soil are taken into account by a modulus of subgrade reaction, which is obtained from in situ test results of the foundation material (Part IV) and used in a direct stiffness analysis.

55. A schematic presenting the geometry of a particular interior monolith at Sandy Lake Dam is presented in Figure 29. Five load cases as follows were analyzed.

- a. Normal Operation
- b. Normal Operation with Truck Loading (H15-44)
- c. Normal Operation with Earthquake
- d. Normal Operation with Ice
- e. High-Water Condition.

56. The applied loads and moments on the pile system are presented in Figures 30-35. Two upper and lower pool combinations were checked at Sandy Lake Dam to be sure that a higher tailwater condition, creating more uplift, would not cause a more critical combination of loading. It did not.

57. A summary of the forces and moments on the piling system obtained by conventional stability analysis is presented in Tables 8 and 9. Table 8 gives results for the 10 pile foundation and Table 9 gives the results with the two upstream piles neglected.

58. At this point, the adequacy of the pile foundation could not be evaluated because allowable vertical and horizontal loads must be known to judge the adequacy of the piles. These allowables were not known for Sandy Lake Dam.

59. To determine the adequacy of the stability of the pile foundation, in situ testing was performed to determine the supporting characteristics of the foundation material. The variation of the subgrade modulus with depth and deformation was obtained. The modulus of subgrade reaction did not show a definite variation with depth (Figures 21 and 28). A conservative constant value, 2000 psi/in., was used in evaluating the pile-foundation system. This value was decreased to 1200 psi/in. due to pile spacing. The reduction factor was calculated by the following formula as suggested by Davisson (1970).

$$h_a = 0.15 \frac{a}{B} - 0.2 \qquad \left(3 < \frac{a}{B} < 8\right) \tag{5}$$

where

h_a = reduction factor

a = center to center pile spacing from upstream to downstream

B = pile diameter.

If the piling layout is adequate for this analysis, the total dam can be considered adequate in stability.

60. The allowable load on the pile was determined based on the material properties of the pile. The stability of the pile foundation was evaluated based on pile loads and the deflections at the top of the pile.

61. The adequacy of the pile foundation was evaluated by determining the adequacy of the following components:

a. Axial, shear, and moments in the pile when compared to allowables based on the properties of the Norway Pine material.

b. The connection of the piling to the structure is assumed to be capable of carrying as much shear load as the pile.

c. The deflections of the piles are evaluated against an allowable of 1/4 in.

Pile Foundation Analysis Using In Situ Soil-Foundation Properties

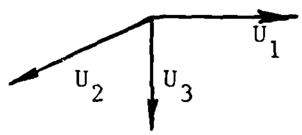
62. A general, direct stiffness analysis for a three-dimensional pile foundation was used as has been presented by Saul (1968), which

expands the Hrennikoff (1950) method from two dimensions to three. The general solution by this stiffness analysis is presented below:

63. The forces on a single pile can be equated to the pile displacements by the expression:

$$\{F\}_i = \{b\}_i \{X\}_i \tag{6}$$

The $\{b\}_i$ values are the individual pile stiffness influence coefficients, called the elastic pile constants. The positive coordinate system is as follows:



The $\{b\}_i$ matrix for a three-dimensional system can be defined for the ith pile as:

$$\{b\}_i = \begin{bmatrix} b_{11} & 0 & 0 & 0 & b_{15} & 0 \\ 0 & b_{22} & 0 & b_{24} & 0 & 0 \\ 0 & 0 & b_{33} & 0 & 0 & 0 \\ 0 & b_{42} & 0 & b_{44} & 0 & 0 \\ b_{51} & 0 & 0 & 0 & b_{55} & 0 \\ 0 & 0 & 0 & 0 & 0 & b_{66} \end{bmatrix}$$

64. The elastic pile constants have meaning as follows:

b_{11} = the force required to displace the pile head a unit distance along the U_1-axis, FORCE/LENGTH

b_{22} = the force required to displace the pile head a unit distance along the U_2-axis, FORCE/LENGTH

b_{33} = the force required to displace the pile head a unit distance along the U_3-axis, FORCE/LENGTH

b_{44} = the moment required to displace the pile head a unit rotation around the U_1-axis, FORCE-LENGTH/RADIAN

b_{55} = the moment required to displace the pile head a unit rotation around the U_2-axis, FORCE-LENGTH/RADIAN

b_{66} = the torque required to displace the pile head a unit rotation around the U_3-axis, FORCE/RADIAN

b_{15} = the force along the U_1-axis caused by a unit rotation of the pile head around the U_2-axis, FORCE/RADIAN

$-b_{24}$ = the force along the U_2-axis caused by a unit rotation of the pile head around the U_1-axis, FORCE/RADIAN (NOTE: The sign is negative.)

b_{51} = the moment around the U_2-axis caused by a unit of displacement of the pile head along the U_1-axis, FORCE-LENGTH/LENGTH.

$-b_{42}$ = the moment around the U_1-axis caused by a unit displacement of the pile head along the U_2-axis, FORCE-LENGTH/LENGTH.

65. Pile i may be located in the foundation with axis through its origin parallel to the foundation axis. The foundation loads $\{Q\}$ and displacements $\{\Delta\}$ are located with respect to the foundation axis.

66. The forces $\{F\}_i$ due to the pile loads on the pile cap are in equilibrium with a set of forces $\{q\}_i$ at the coordinate center of the pile cap.

67. Equilibrium yields

$$\{q\}_i = \{C\}_i \{F\}_i \tag{7}$$

in which $\{C\}_i$, the statics matrix for a three-dimensional system, is

$$\{C\}_i = \begin{bmatrix} 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & -U_3 & U_2 & 1 & 0 & 0 \\ U_3 & 0 & -U_1 & 0 & 1 & 1 \\ -U_2 & U_1 & 0 & 0 & 0 & 1 \end{bmatrix}$$

where

$u_1 = U_1$ coordinate of the pile, LENGTH

$u_2 = U_2$ coordinate of the pile, LENGTH

$u_3 = U_3$ coordinate of the pile, LENGTH

Foundation stiffness analysis

68. If the piling cap is assumed rigid, the deflection of the pile cap can be related to the deflection of the piling in the foundation axis coordinates by

$$\{X\}_i = \{C\}_i^T\{\Delta\} \tag{8}$$

69. The foundation load $\{Q\}$ is distributed to each piling so that

$$\{Q\} = \sum_{i=1}^{n} \{q\}_i \tag{9}$$

where n = number of piles . The relationships between the foundation load and the pile cap deflections are

$$\{Q\} = \{S\}\{\Delta\} \tag{10}$$

in which $\{S\}$ is the stiffness influence coefficients matrix for the foundation as a whole. The $\{S\}$ matrix is found by introducing the contribution of each individual pile toward the stiffness of the pile cap. This yields

$$\{q\}_i = \{S'\}_i\{\Delta\} \tag{11}$$

in which

$$\{S'\}_i = \{C\}_i\{a\}_i\{b\}_i\{a\}_i^T\{C\}_i^T \tag{12}$$

and finally

$$\{S\} = \sum_{i=1}^{n} \{S'\}_i \tag{13}$$

where $\{a\}$ is the transformation matrix of force and displacement of the pile (rotated and/or battered) axis to the foundation axis.

70. Once the stiffness matrix is known for the total foundation, the problem is essentially solved and requires only back substitution to find the distribution of loads to the individual piling. It should be noted that the foundation stiffness matrix $\{S\}$ is independent of the external loads.

Loads and displacements

71. The displacement of the pile cap can be found by inverting the foundation stiffness matrix $\{S\}$ and multiplying it by the external load matrix $\{Q\}$ or

$$\{\Delta\} = \{S\}^{-1}\{Q\} \tag{14}$$

72. Once the foundation deflections are known, the deflections of pile i about its own axis can be found by

$$\{X\}_i = \{a\}_i^T\{C\}_i^T\{\Delta\} \tag{15}$$

73. Finally, the forces allotted to each pile about its axes can be found from Equation 6 where

$$\{F\}_i = \{b\}_i\{X\}_i \qquad \text{(6 bis)}$$

Forces and Deflections of Individual Piles

74. The approach followed in obtaining the forces and deflections on the individual piles was as follows. The modulus of reaction, the material properties of the pile, and the pile length were used to determine the pile-head stiffness matrix for a single pile, assuming a linear elastic pile-soil system. This pile-head stiffness matrix was obtained by using a finite element computer code (Marlin, Jones, and Radhakrishnan, in preparation), which is a one-dimensional finite element analysis of a beam on an elastic foundation.

75. The pile-head stiffness matrix was then used as input in another computer program (LMVD PILE, WES Technical Report K-80-3) that

uses the direct stiffness analysis to obtain the forces and deflections of the piles. A beam on an elastic foundation analysis was also performed and the pressures, moments, and deflections along the length of the most critically loaded pile were determined.

76. The analysis assumed that the top of the pile was pinned to the base of the monolith, and that the monolith base is rigid. These assumptions are adequate for the dam construction of Sandy Lake Dam.

77. The results of the three-dimensional, direct stiffness pile foundation analysis are presented in Tables 10 and 11. The allowable load on the pile are as follows:

Maximum allowable compressive load per pile = 124 kips
Maximum allowable shear load per pile = 8.5 kips
Maximum allowable tensile load per pile = 0 kips
Maximum allowable moment in a pile = 131,000 in.-lb or 10.9 kip-ft

78. The pressures, moments, and deflections along the length of the pile for the most critical load case (high-water condition) is presented in Figure 38. The maximum and minimum stresses in the pile due to the applied loads and moments are 950 psi compression and 580 psi tension.

79. The stresses in the pile and the axial load on any pile are well below the allowables based on the strength of the pile. However, the shear loads are above the allowables. The allowable horizontal load of 8.5 kips per pile is reasonable; therefore, it is advisable to provide for at least a downstream strut resistance of 40 kips per pier.

80. There was some tension in the row of upstream piles when 10 piles were used. When the two extreme upstream piles were neglected, there was no tension on the remaining piles, which makes the pile foundation adequate as far as tensile loads are concerned.

81. The horizontal and vertical deflections of the piles were below the allowable of 1/4 in. and are acceptable (moments of inertia are presented in Figures 36 and 37).

82. The pile layout was close to being symmetrically located under the pier. From a consideration of the pile layout, the applied

loads, and the previous analysis, it can be seen that the foundation will be adequate for some reversal in head due to backwaters from the Mississippi River.

83. Since there are no noticeable vertical deflections of the structure and due to the adequacy of the foundat. n, it is not expected that vertical deflections of the dam monoliths will cause any loss of reservoir pool in the future; therefore, the vertical deflections of piles were computed only as axial deflections $(\frac{PL}{AE})$ to save time and expense.

84. From upstream to downstream, the lock structure walls are stable such that a loss in pool is not a problem. For this reason the stability of the lock structure is not critical and was not analyzed.

PART VI: CONCLUSIONS AND RECOMMENDATIONS

Foundation

85. The soil-piling system that supports the piers at Sandy Lake Dam is adequate except for the safety of the piles in their resistance to shear loads. It is recommended that 40 kips of strut resistance be provided downstream of each of the four interior small piers. The strut resistance could be obtained by assuring positive resistance from the piers to the downstream apron.

Concrete

86. The interior concrete is of good quality, but the surface concrete is deteriorating. The concrete surface needs to be rehabilitated to ensure that water is not allowed to enter cracks and accelerate the deterioration of the concrete.

87. There are a number of methods of rehabilitation that might be used. If an overlay coating is used, it must allow the concrete to "breathe" (allow water to escape from the interior concrete through the coating). Since the concrete is not air-entrained, it cannot resist freezing and thawing if frozen while in a saturated condition. It is therefore essential that water be able to escape from the interior concrete through the coating.

88. After the surface rehabilitation has been accomplished, the structure should be adequate for many more years of service.

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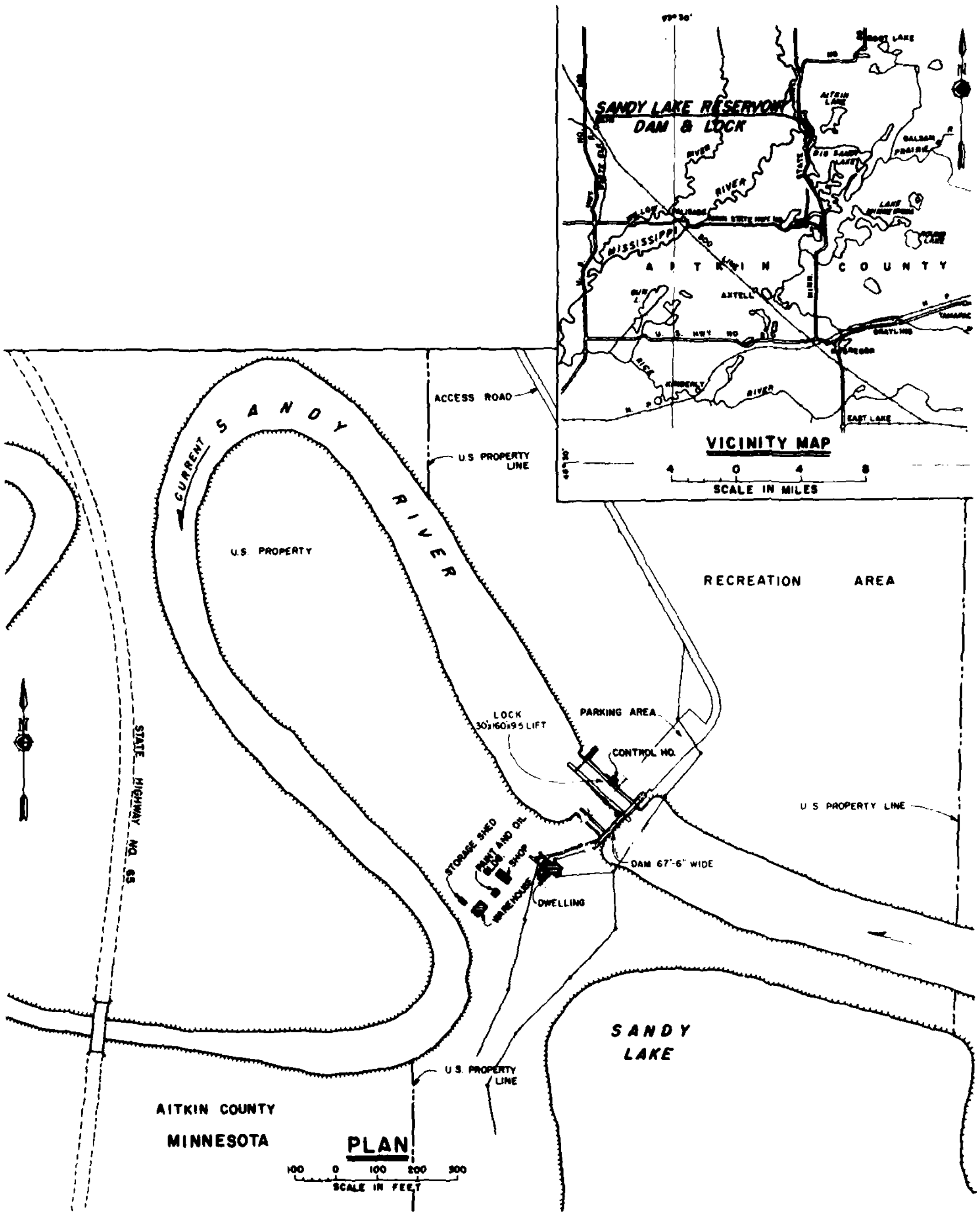

Figure 1. Plan and vicinity maps of Sandy Lake Dam area

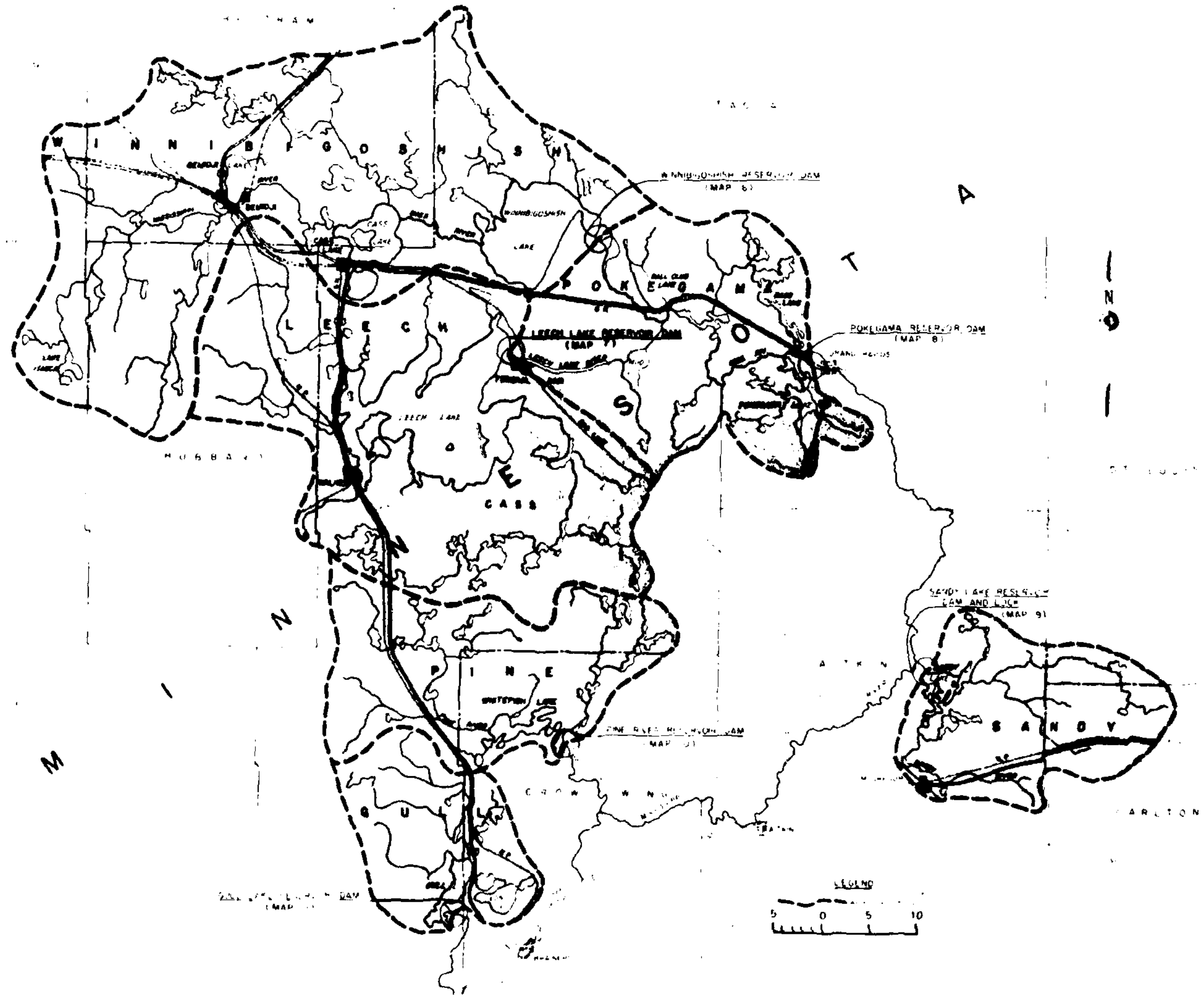

Figure 2. Project map of Mississippi River Headwaters reservoirs

a. Aerial site photo

b. Downstream view of structure

Figure 3. Photos of Sandy Lake Dam

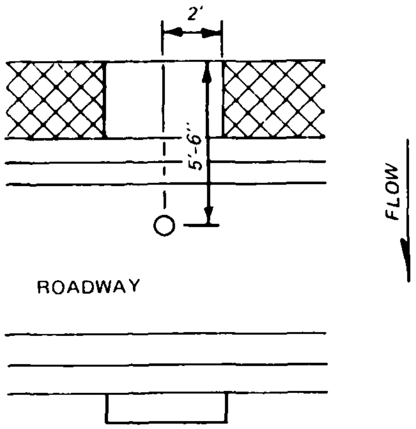

Figure 4. Location of core holes, piers 1 and 5, Sandy Lake Dam

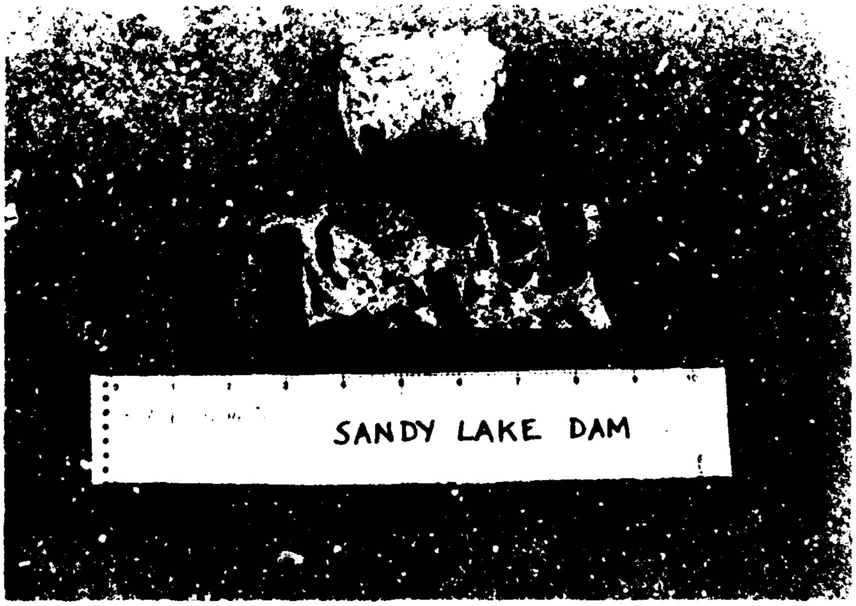

Figure 5. Representative concrete core and cut section

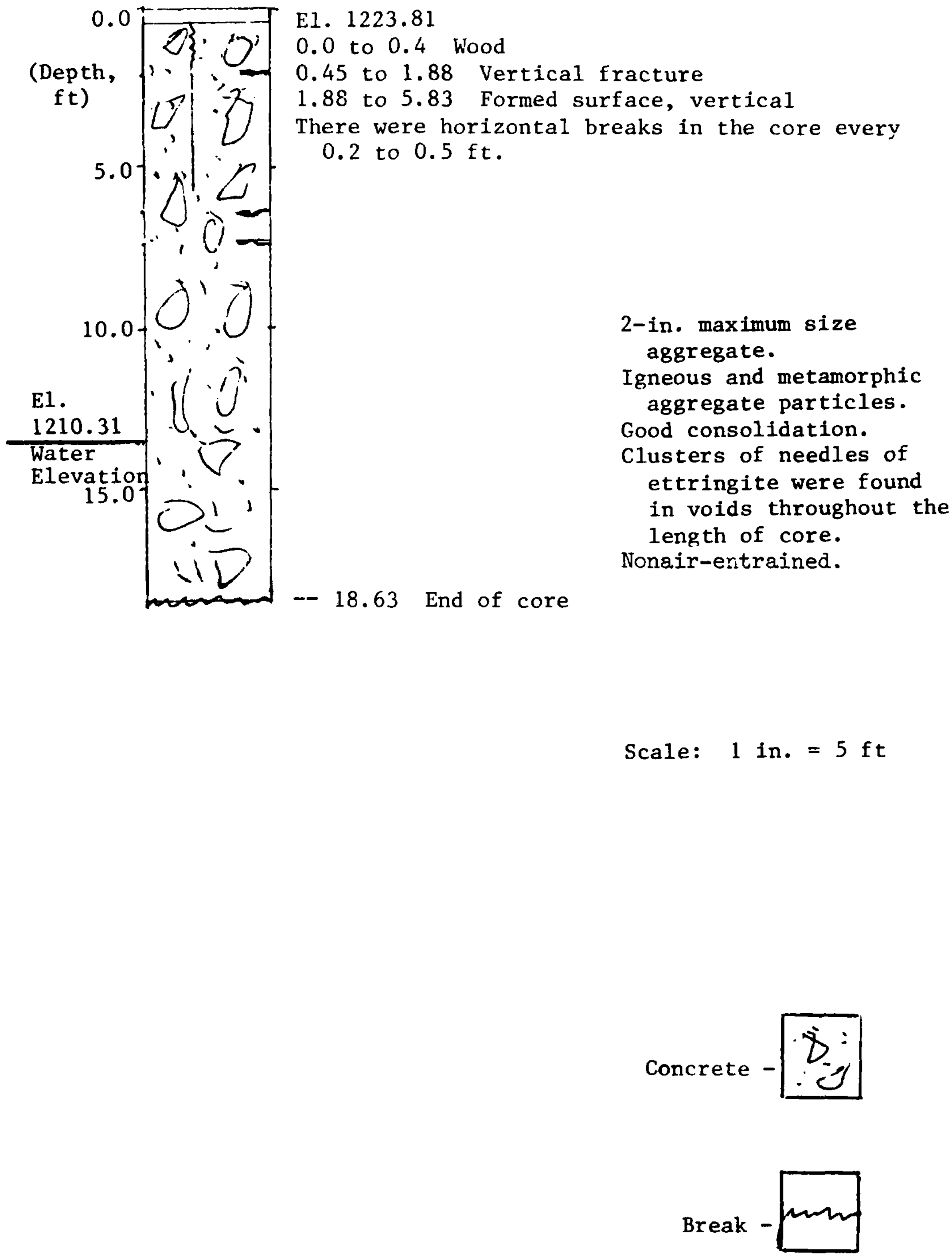

Figure 6. Vertical NX concrete core, S-P1, Sandy Lake Dam

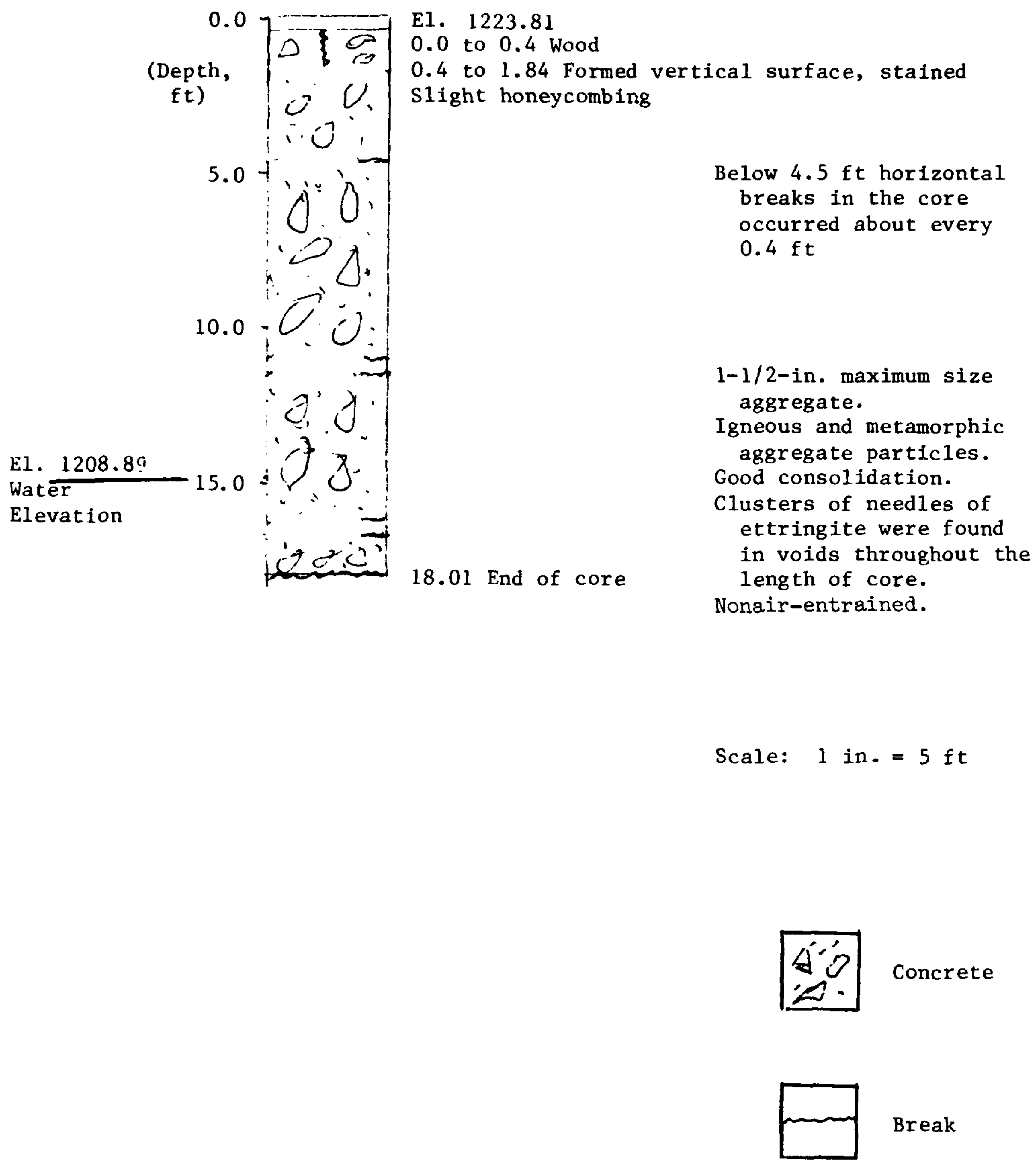

Figure 7. Vertical NX concrete core, S-P5, Sandy Lake Dam

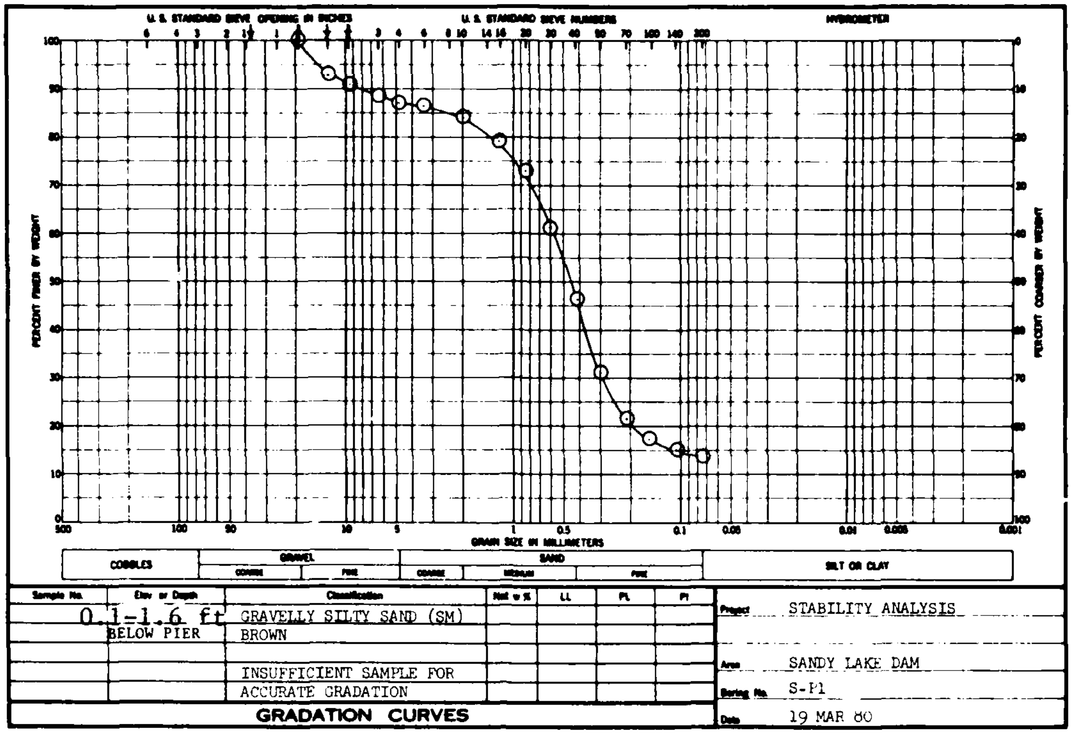

Figure 8. Foundation soil sieve analysis and classification, 0.1-1.6 ft

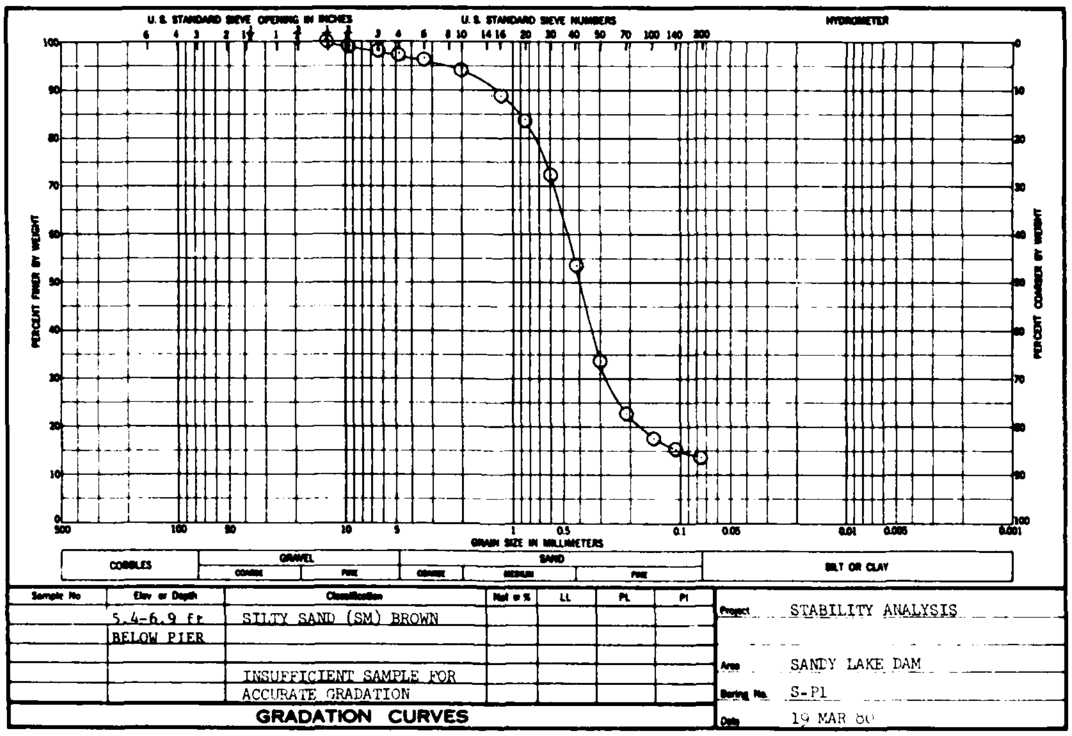

Figure 9. Foundation soil sieve analysis and classification, 5.4-6.9 ft

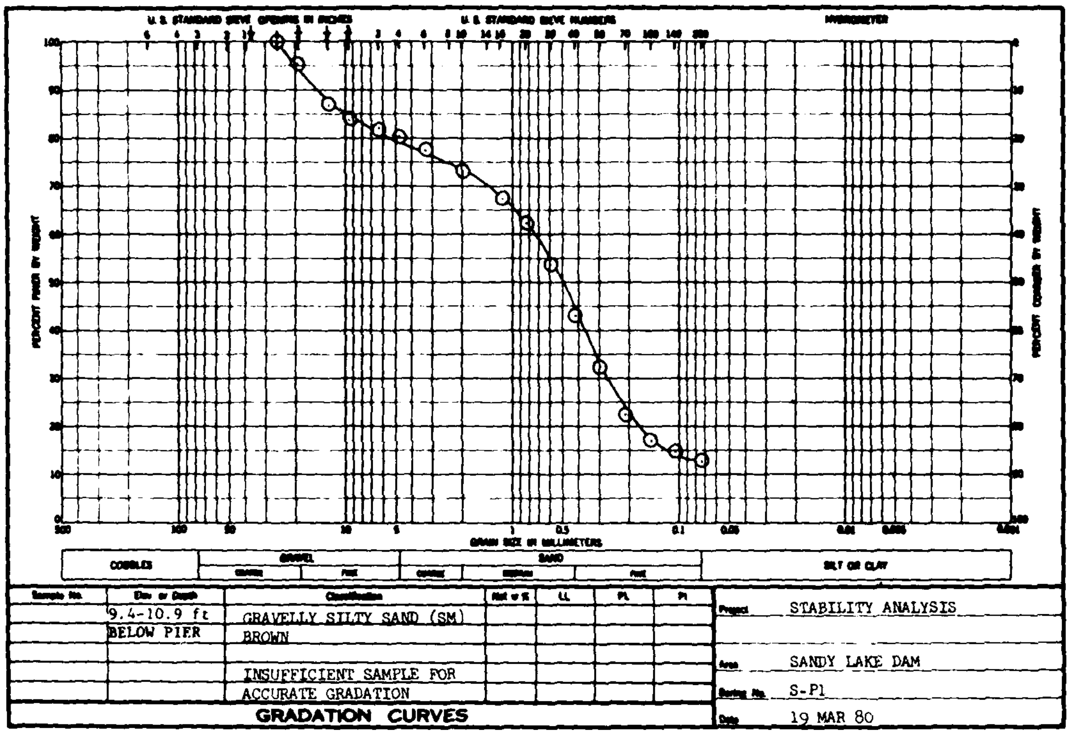

Figure 10. Foundation soil sieve analysis and classification, 9.4-10.9 ft

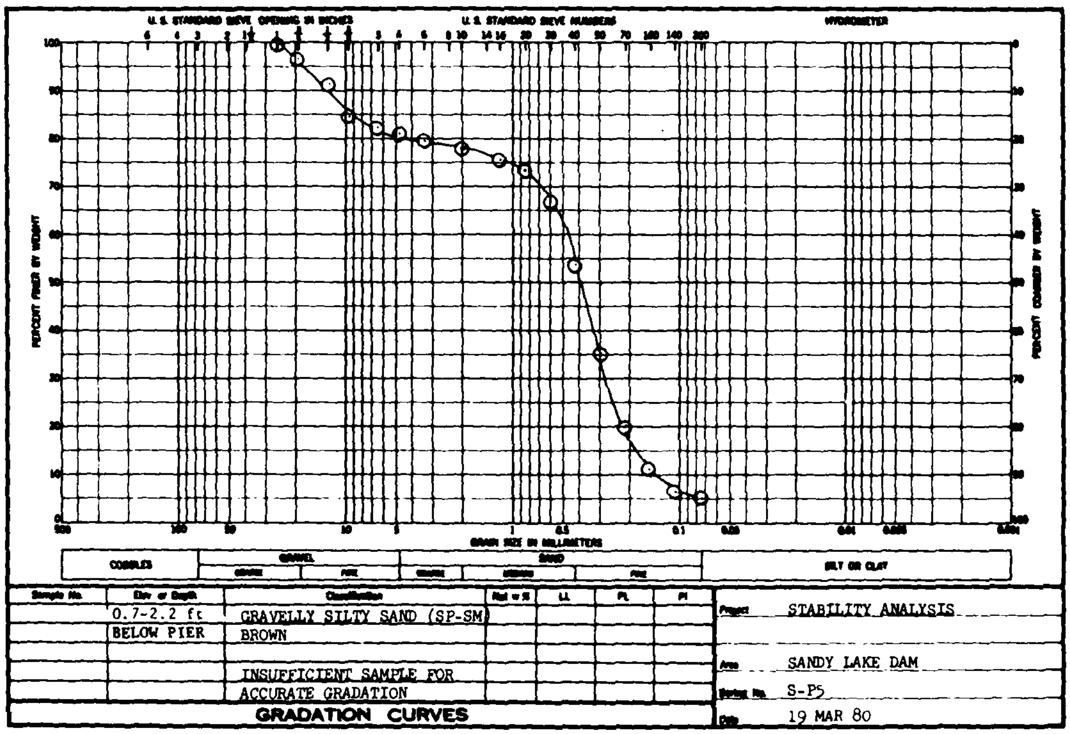

Figure 11. Foundation soil sieve analysis and classification, 0.7-2.2 ft

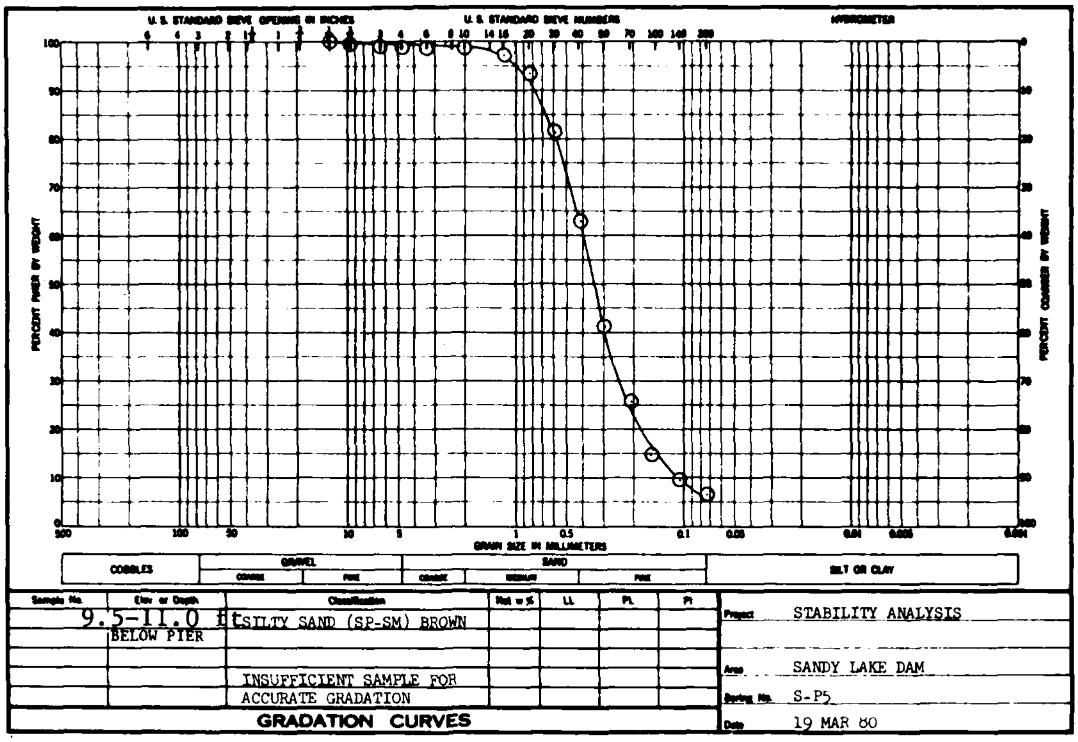

Figure 12. Foundation soil sieve analysis and classification, 9.5-11.0 ft

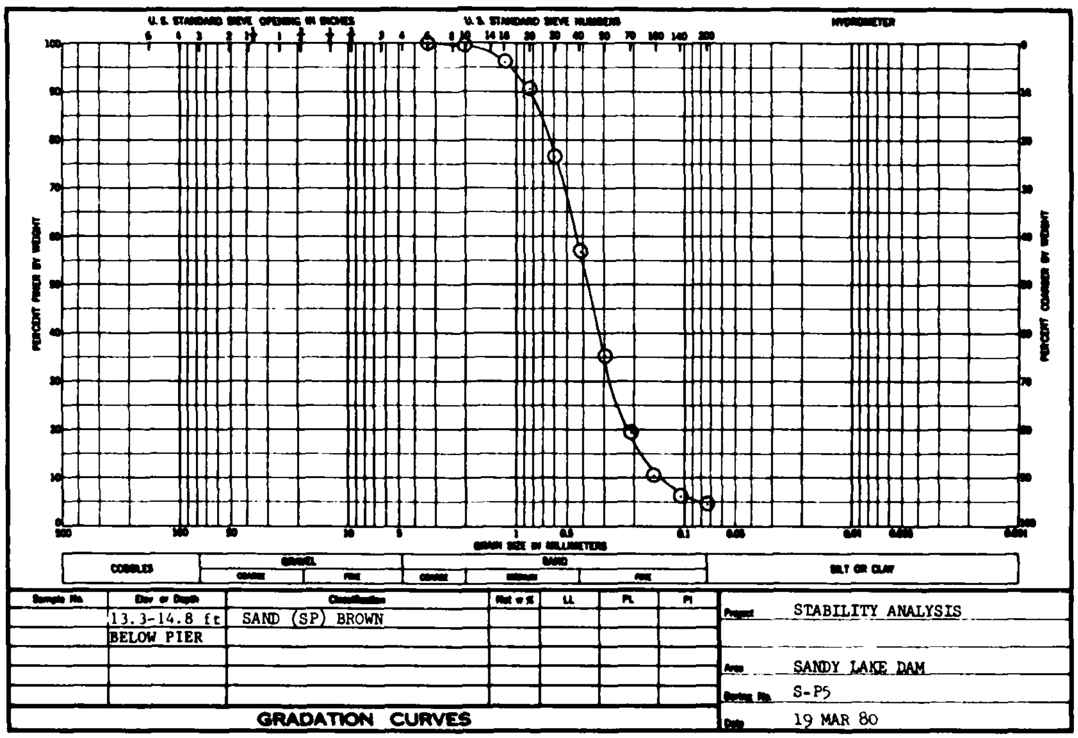

Figure 13. Foundation soil sieve analysis and classification, 13.3-14.8 ft

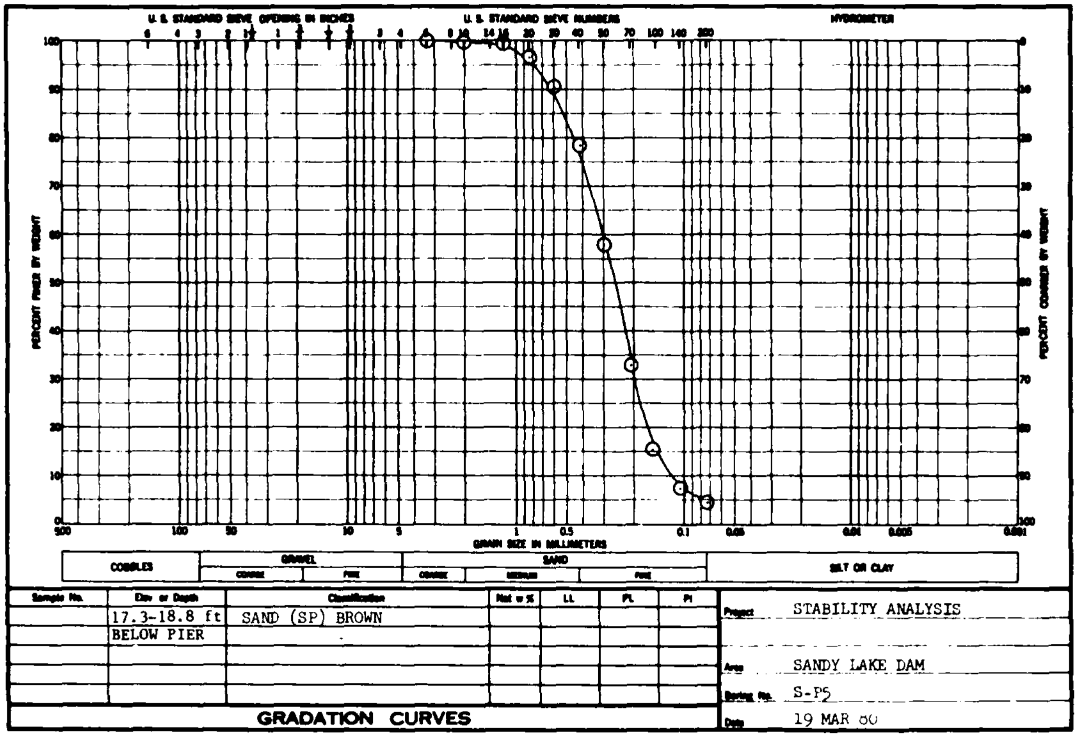

Figure 14. Foundation soil sieve analysis and classification, 17.3-18.8 ft

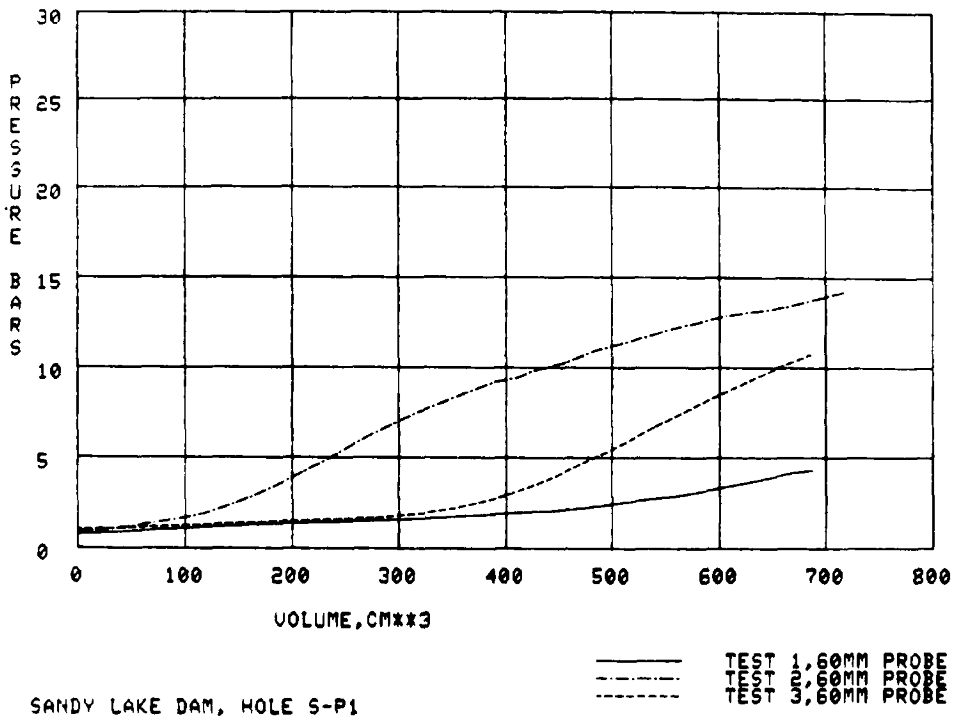

Figure 15. Pressure versus volume (metric units), S-P1

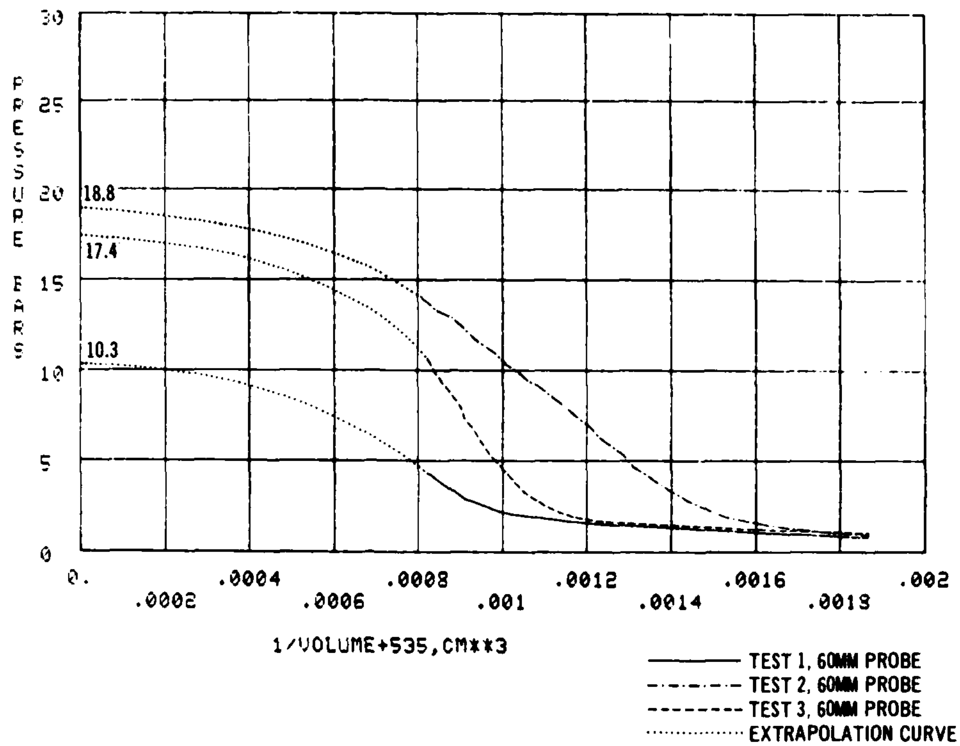

Figure 16. Limit pressure determination with curves extrapolated, S-P1

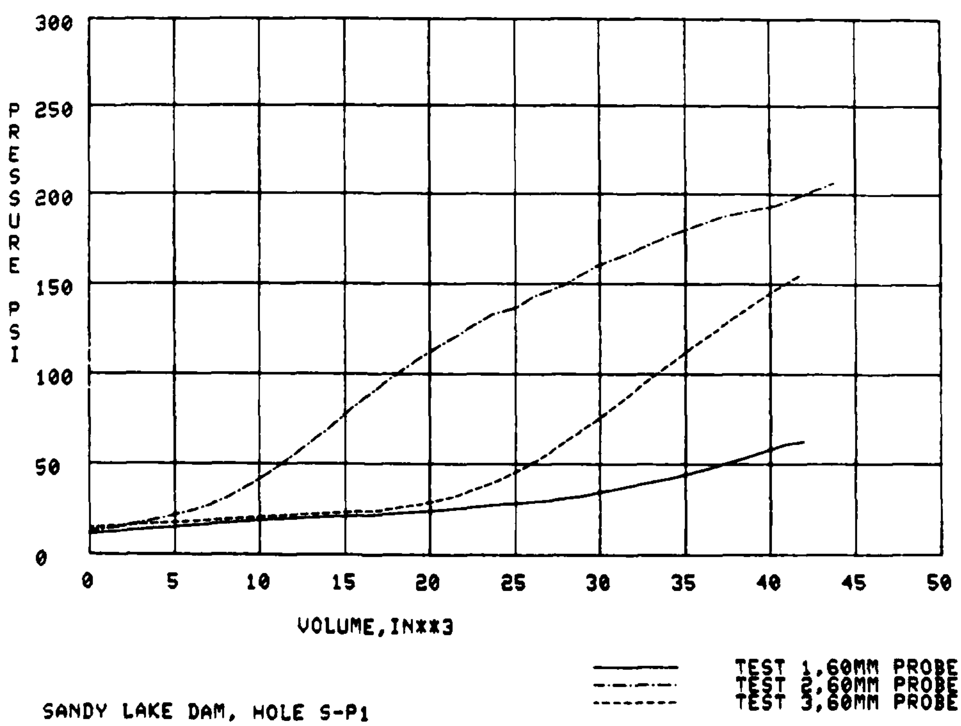

Figure 17. Pressure versus volume, S-P1

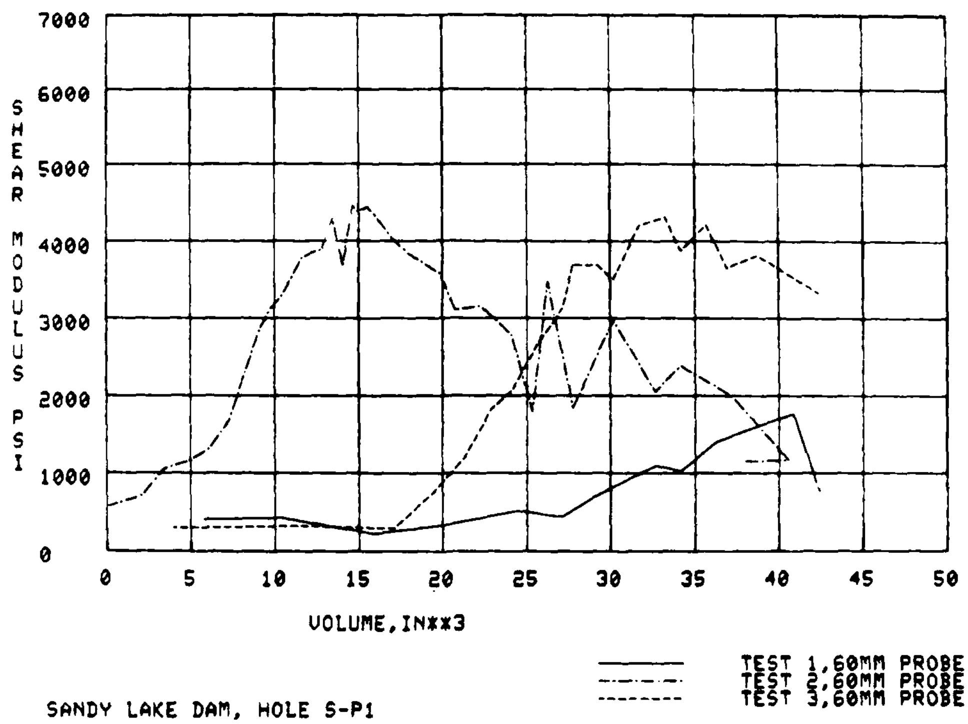

Figure 18. Shear modulus, S-P1

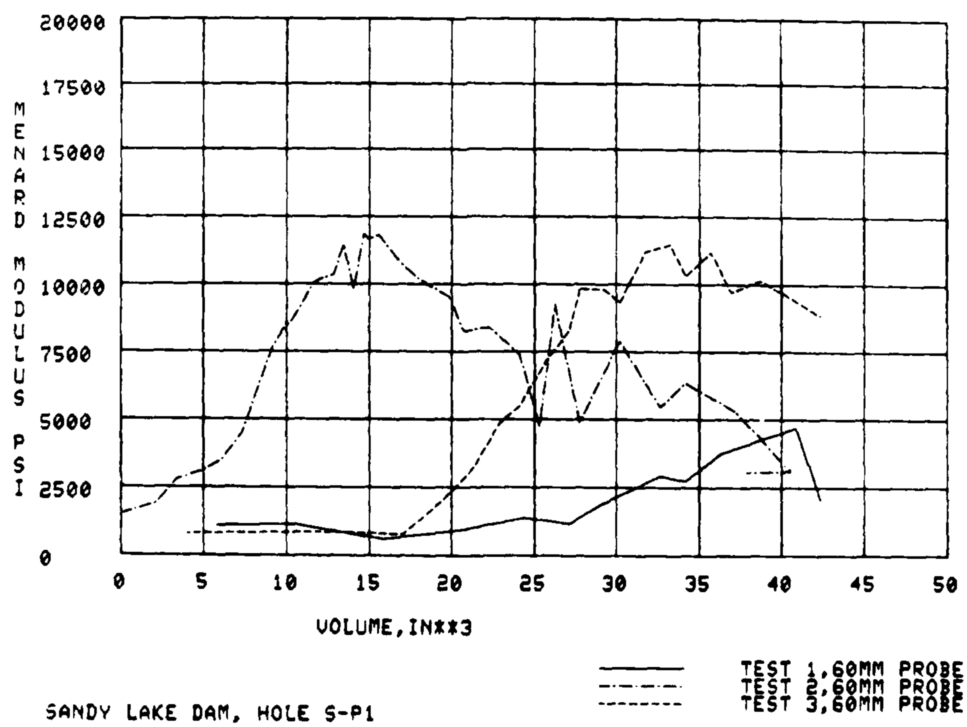

Figure 19. Ménard modulus, S-P1

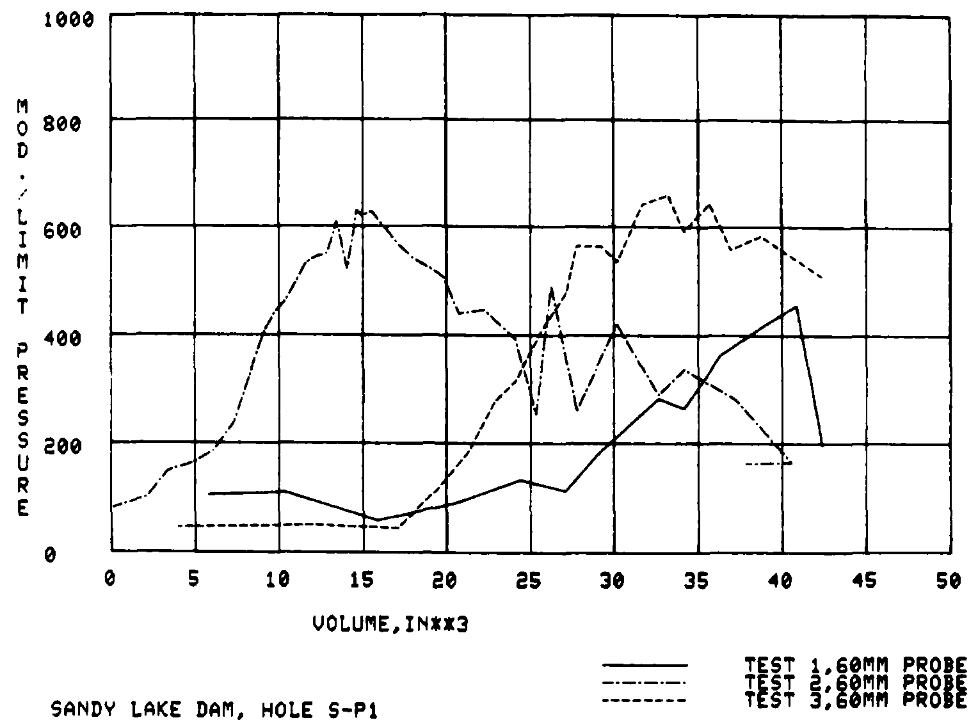

Figure 20. Ménard modulus divided by limit pressure, S-P1

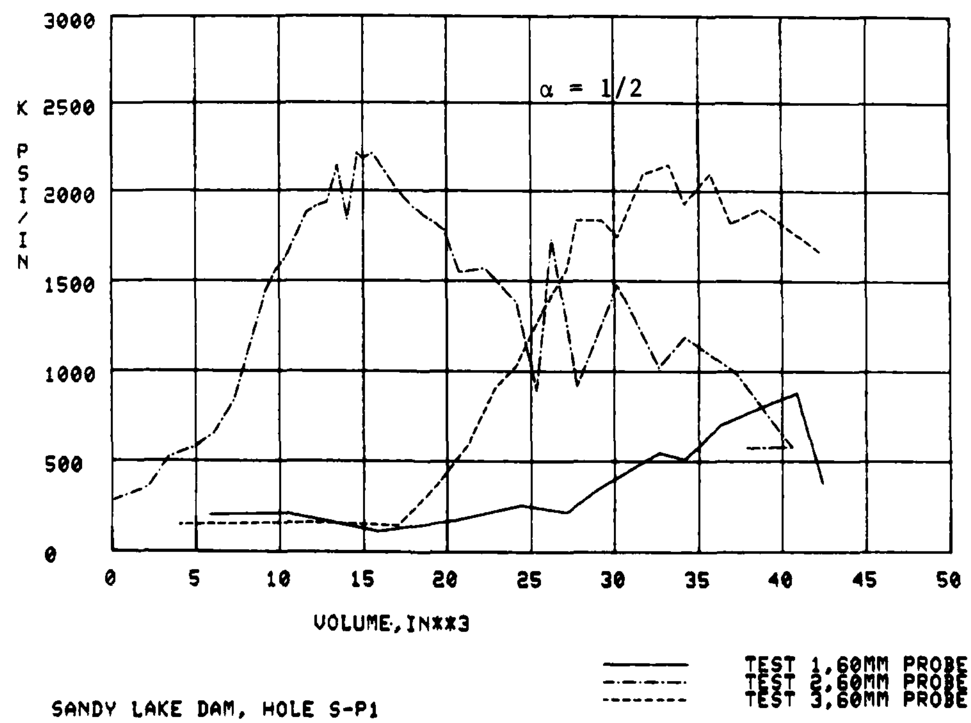

Figure 21. Modulus of subgrade reaction, S-P1

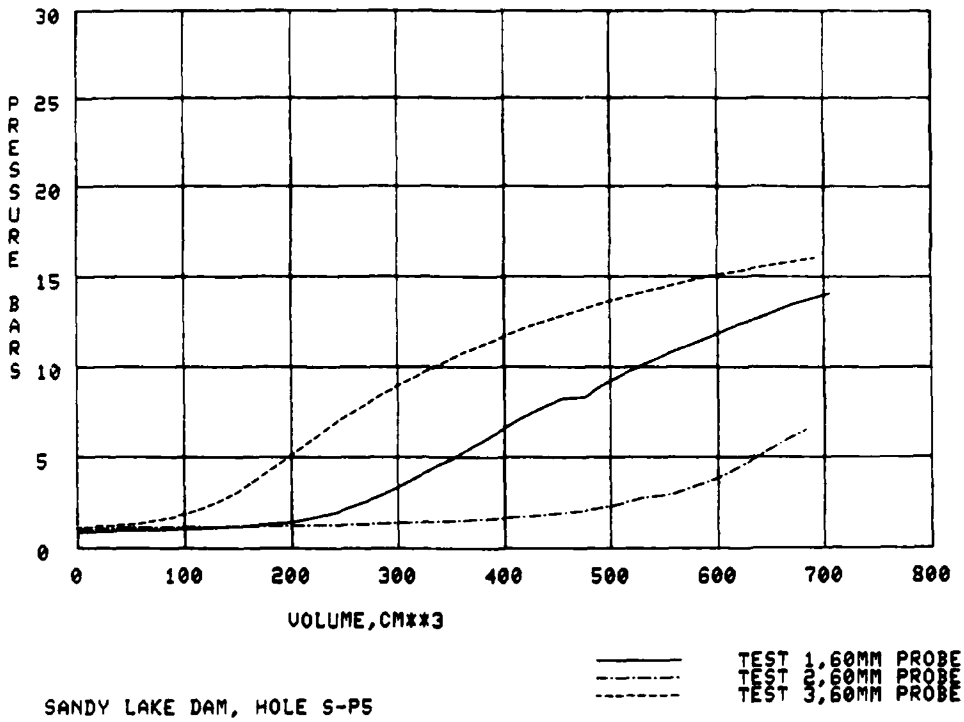

Figure 22. Pressure versus volume (metric units), S-P5

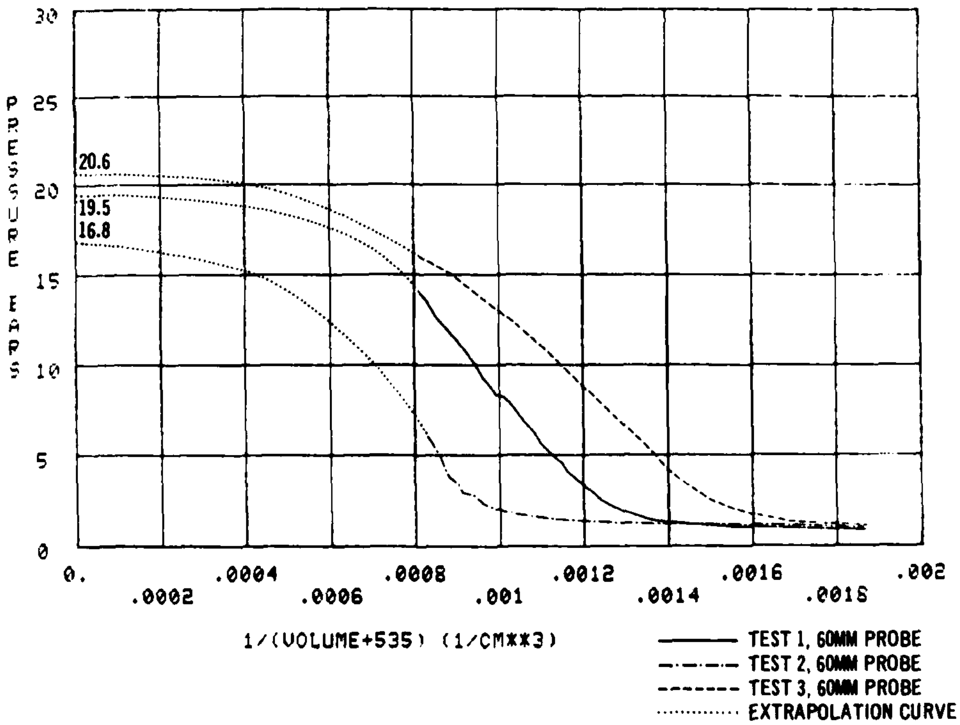

Figure 23. Limit pressure determination with curves extrapolated, S-P5

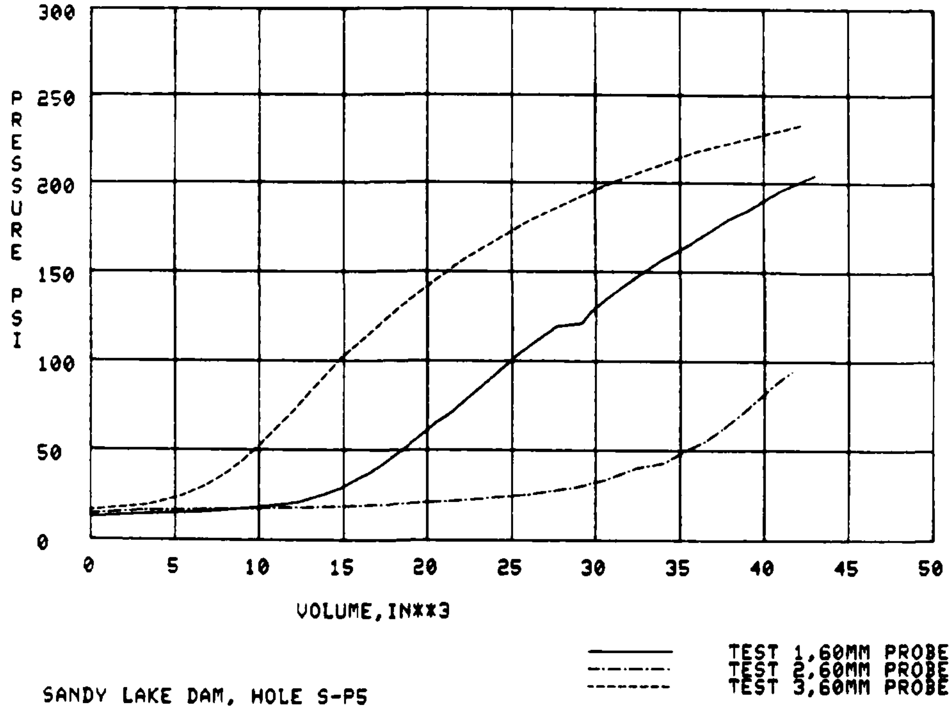

Figure 24. Pressure versus volume, S-P5

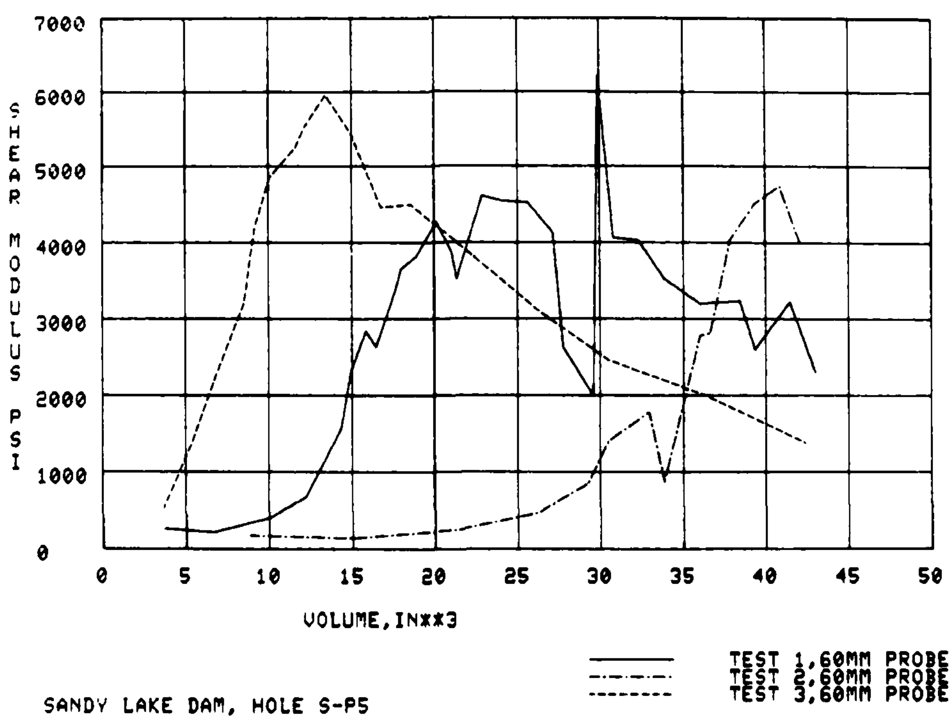

Figure 25. Shear modulus, S-P5

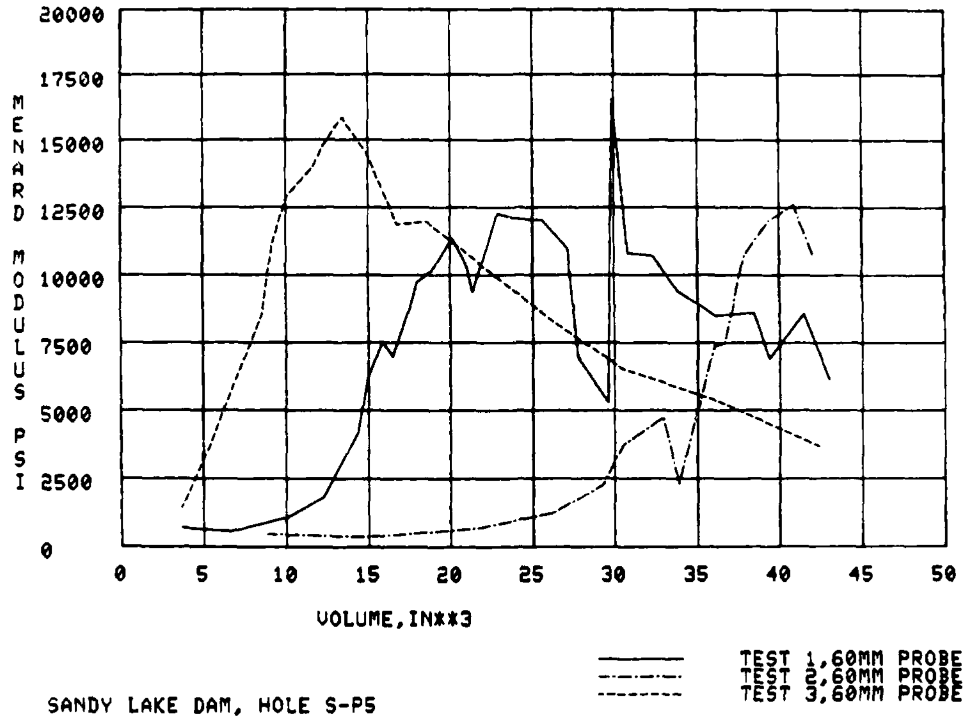

Figure 26. Ménard modulus, S-P5

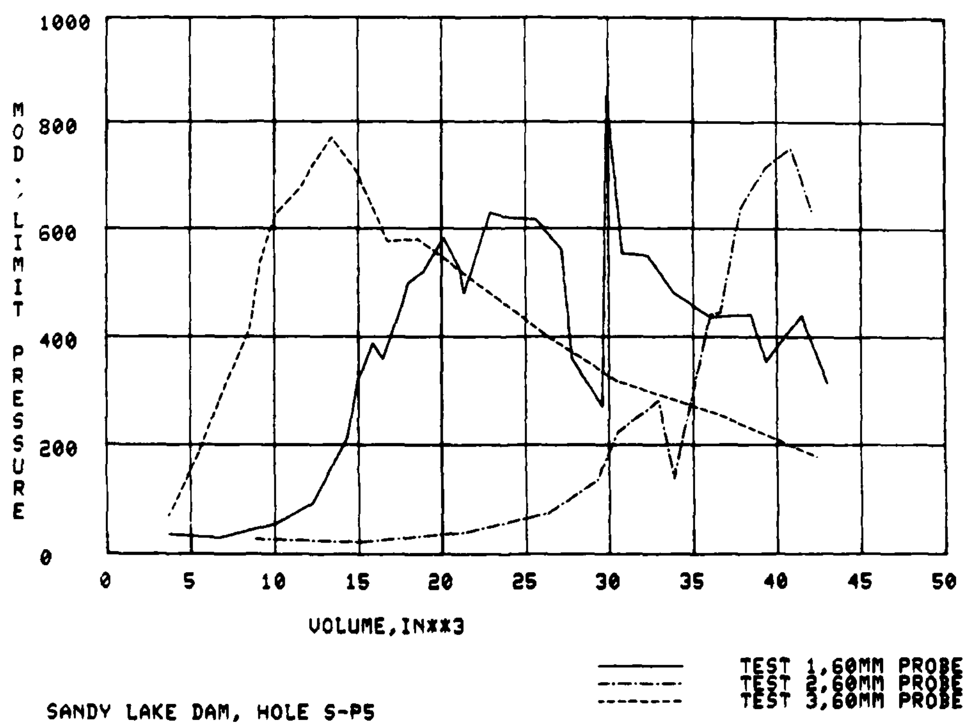

Figure 27. Ménard modulus divided by limit pressure, S-P5

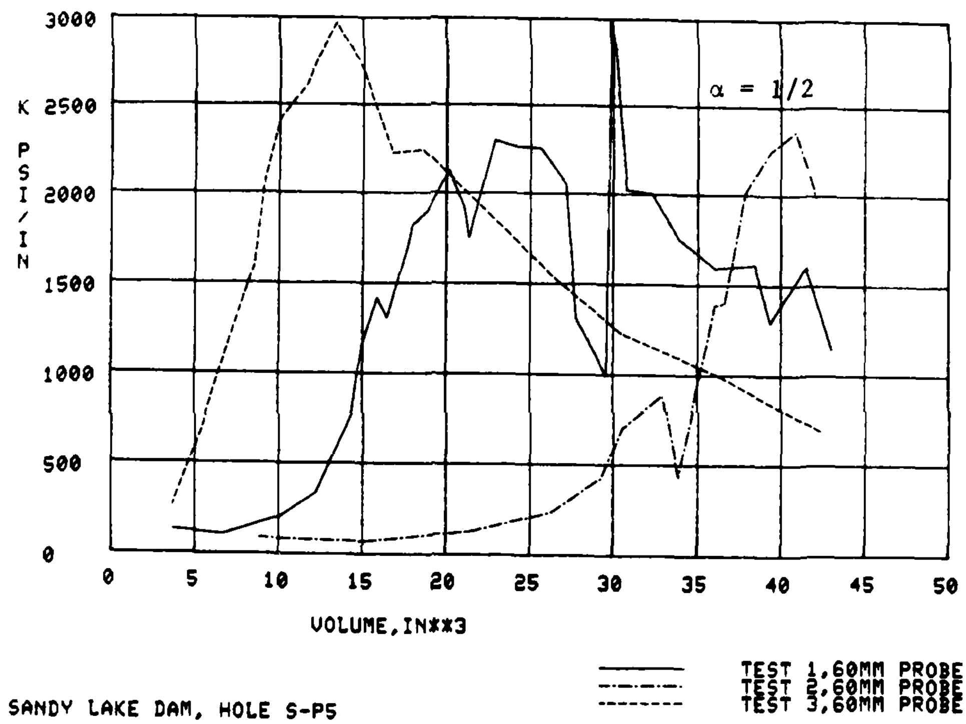

Figure 28. Modulus of subgrade reaction, S-P5

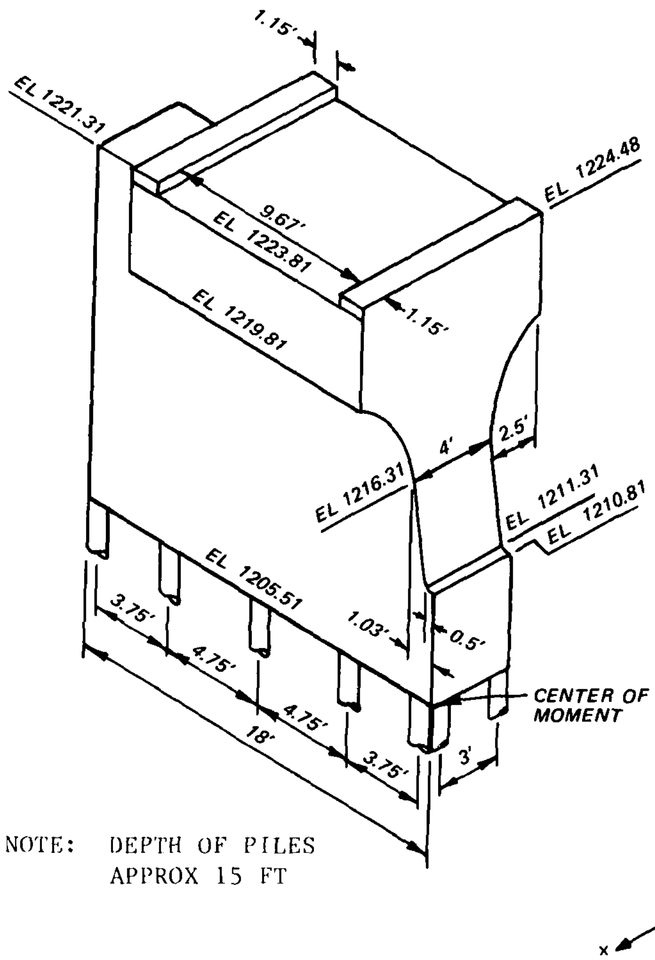

Figure 29. Schematic of a Sandy Lake Dam section

Item	Factor	F_H (kips)	F_V (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-kip)
W_{Conc}	(0.15)(2)(1.15)(1224.51 - 1223.81)(9)		2.17	7.02		15.2
	(0.15)[(2)(1.15) + 9.67][1223.81 - 1219.81)(9)]		64.64	7.02		453.8
	$(0.15)[(2)(1.15) + 9.67][(2.5)(5) - \frac{\pi(2.5)^2}{2}]$		4.82	7.02		33.8
	(0.15)[5 + (2)(1.15) + 9.67][1219.81 - 1205.51](4)		145.60	9.52		1386.1
	(0.15)[18 - 5 - (2)(1.15) - 9.67)][1210.81 - 1205.51](4)		3.28	0.52		1.7
	(0.15)(1/2)[18 - 5 - (2)(1.15) - 9.67 - 0.5] [1216.31 - 1210.81](4)		0.87	0.85		0.7
			221.38			1891.3
$F_{Headwater}$	$(0.0625)(1/2)(1218.31 - 1205.51)^2(9)$	-46.08			4.27	-196.8
$F_{Tailwater}$	$(0.0625)(1/2)(1206.31 - 1205.5)^2(9)(0.6)$ (EM 1110-2-2200)	0.11			0.27	0.0
Uplift	-(0.0625)(1218.31 - 1205.51)(4.25)(4)		-13.60	15.88		-216.0
	-(0.0625)(1206.31 - 1205.51)(13.75)(4)		-2.75	6.88		-18.9
	$(0.0625)(1/2)[1218.31 - 1206.31](\frac{13.75}{13+13+13.75})(14)$		-0.52	9.17		-4.8
			-16.87			-239.7
	$e = \frac{1454.8}{204.51} = 7.11$ ft					
Total		-45.97	204.51			1454.8

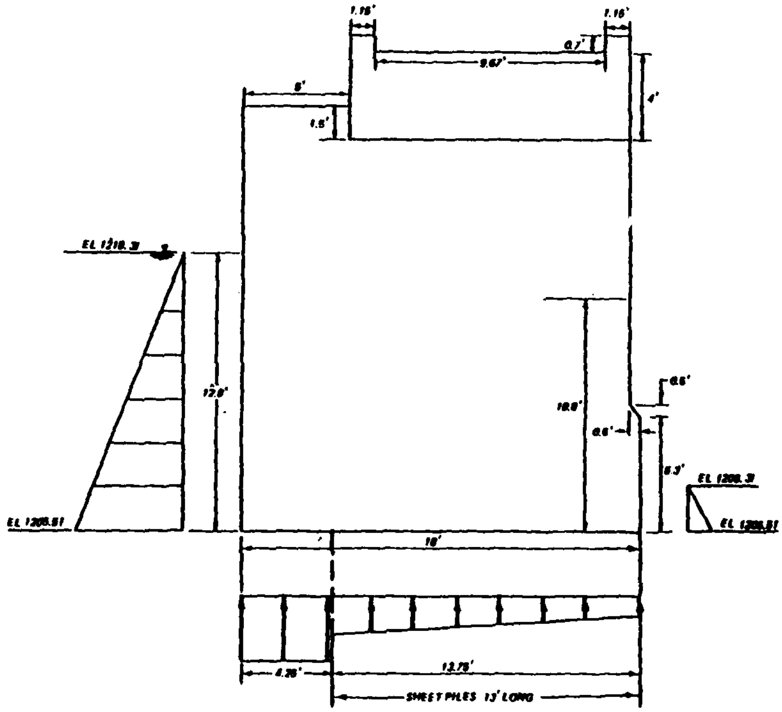

Figure 30. Normal operation case loading, Sandy Lake Dam

Item	Factor	F_H (kips)	F_V (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-kip)
Loads	From Normal Operation Calculations	-45.97	204.51			1454.8
P_{Truck}			24.0		7.02	168.5
	$e = \frac{1623.3}{228.51} = 7.10$ ft					
Total		-45.97	228.51			1623.3

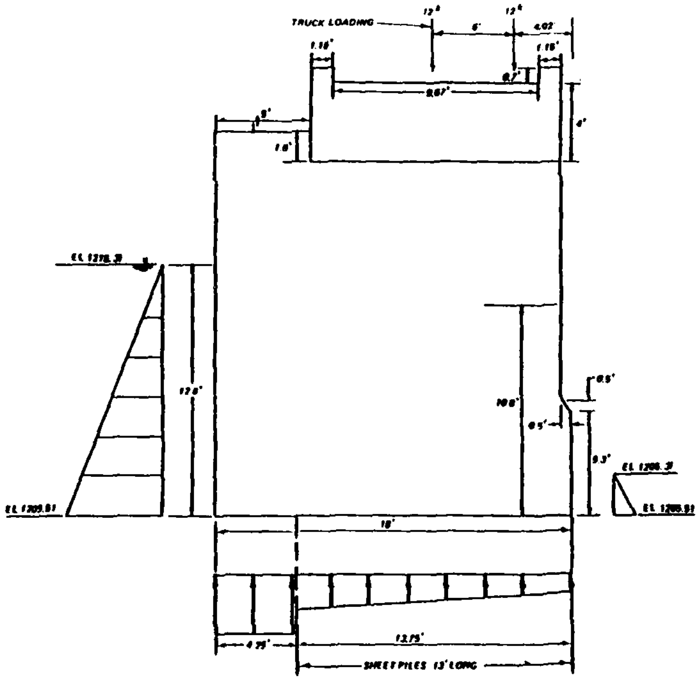

Figure 31. Noraml operation with truck loading (H15-44), Sandy Lake Dam

Item	Factor	F_H (kips)	F_V (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-kip)
Loads	From Normal Operation Calculations	-45.97	204.51			1454.8
Earthquake:						
Pe_1	(0.025)(221.38)	-5.53		10.01		-55.4
Pe_2	$(2/3)(51)(0.025)(1218.31-1205.51)^2(9)(1/1000)$	-1.25		5.12		-6.4
	$e = \frac{1393}{204.51} = 6.81$ ft					
Total		-52.75	204.51			1393.00

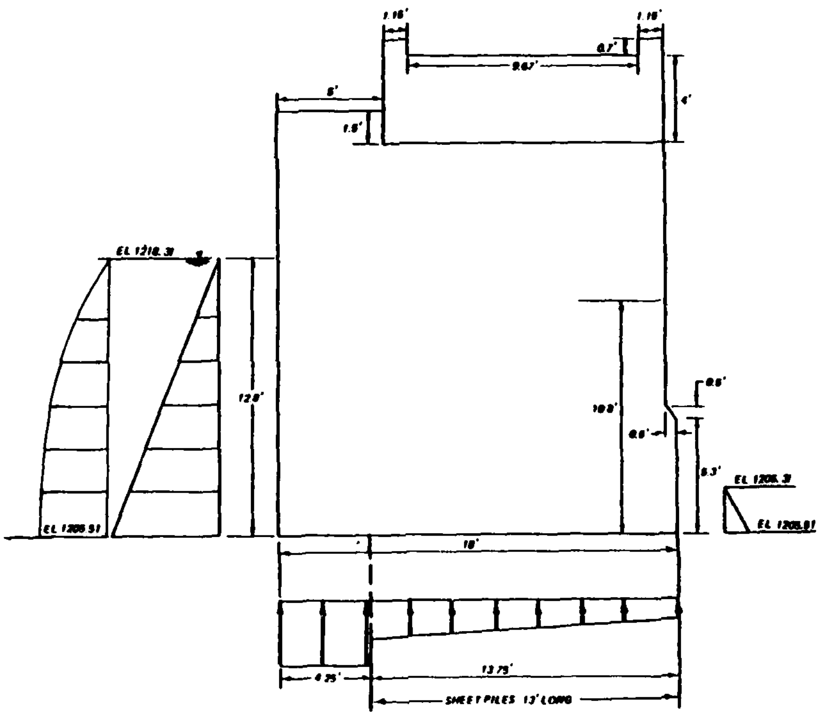

Figure 32. Normal operation with earthquake, Sandy Lake Dam

LOCATION OF CENTROID OF PIER IN XZ PLANE

(2.17)[(1/2)(1224.51 - 1223.81) + 1223.81 - 1205.51]	=	40.5
(64.64)[(1/2)(1223.81 - 1219.81) + 1219.81 - 1205.51]	=	1053.6
(22.44)[(1/2)(1219.81 - 1217.31) - 1217.31 - 1205.51]	=	292.8
$-(17.62)[(\frac{4}{3\pi})(1219.81 - 1217.31) + (1217.31 - 1205.51]$	=	-226.6
(145.60)(1/2)(1219.81 - 1205.51)	=	1041.0
(3.28)(1/2)(1219.81 - 1205.51)	=	8.7
(0.87)[(1/3)(1216.31 - 1210.81) + (1210.81 - 1205.51)]	=	6.2
		2216.2

$$\bar{Z} = \frac{2216.2}{221.38} = 10.01 \text{ ft}$$

Figure 33. Normal operation with earthquake; location of centroid of pier in XZ plane

Item	Factor	F_H (kips)	F_V (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-kip)
Loads	From Normal Operation Calculations	-45.97	204.51			1454.8
P_{ice}	(1)(5)(9) (EM 1110-2-2200)	-45.00		12.30		-553.5
	$e = \frac{901.3}{204.5} = 4.41$ ft					
Total		-90.97	204.51			901.3

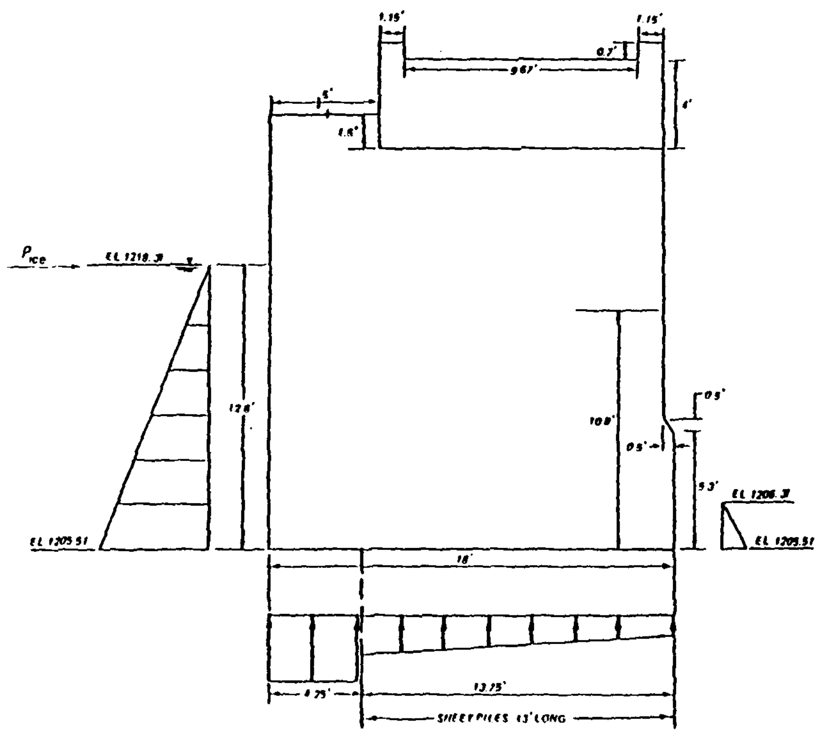

Figure 34. Normal operation with ice loading, Sandy Lake Dam

Item	Factor	F_H (kips)	F_V (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-kip)
W_{Conc}	See Calculations for Normal Operation Condition		221.38			1891.3
$P_{Headwater}$	$(0.0625)(1/2)(1225.31 - 1205.51)^2(9)$	-110.26			6.60	-727.7
$P_{Tailwater}$	$(0.0625)(1/2)(1213.31 - 1205.51)^2(9)(0.6)$	10.27			2.60	26.7
Uplift	$-(0.0625)(1225.31 - 1205.51)(4.25)(4)$		-21.04	15.88		-334.1
	$-(0.0625)(1213.31 - 1205.51)(13.75)(4)$		-26.81	6.88		-184.4
	$-(0.0625)(1/2)[1225.31-1213.31][\frac{13.75}{13+13+13.75}](4)$		-0.52	9.17		-4.8
			-48.37			-523.3
	$e = \frac{667.0}{173.01} = 3.86$ ft					
Total		-99.99	173.01			667.0

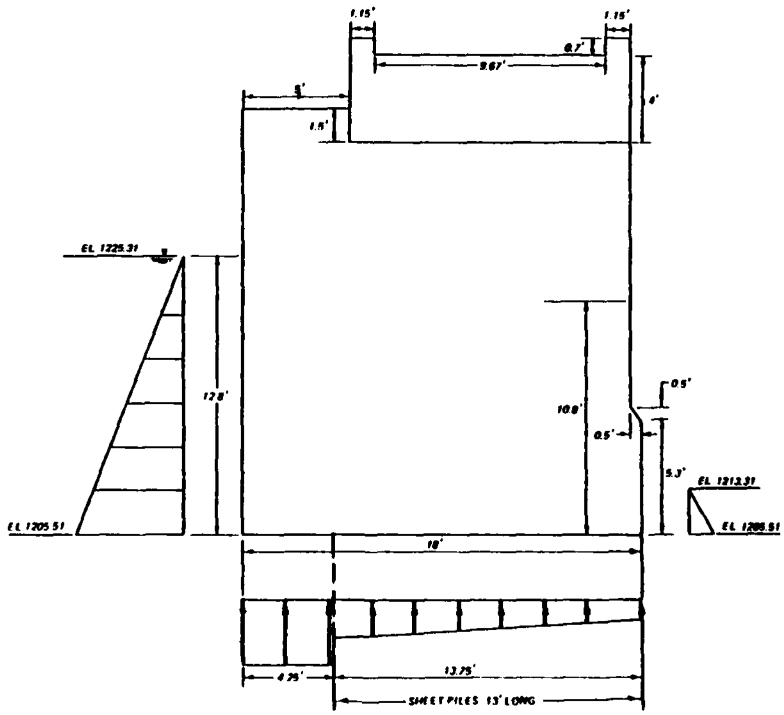

Figure 35. High-water condition, Sandy Lake Dam

LOCATION OF CENTROID OF PILE GROUP

$$\bar{Y} = \frac{(2)[1 + 4.75 + 9.5 + 14.25 + 18.13]}{(10)}$$

$$\bar{Y} = 9.53 \text{ ft}$$

$$X = (1/2)(6) = 3 \text{ ft}$$

MOMENT OF INERTIAL OF PILE GROUP ABOUT CENTROID OF PILE GROUP

$$I = Nd^2$$

$$I_{xx} = (2)[(9.53 - 1)^2 + (9.53 - 4.75)^2 + (9.53 - 9.50)^2 + (9.53 - 14.25)^2 + (9.53 - 18.13)^2]$$

$$I_{xx} = 385 \text{ ft}^4$$

$$I_{yy} = (10)(3)^2$$

$$I_{yy} = 90 \text{ ft}^4$$

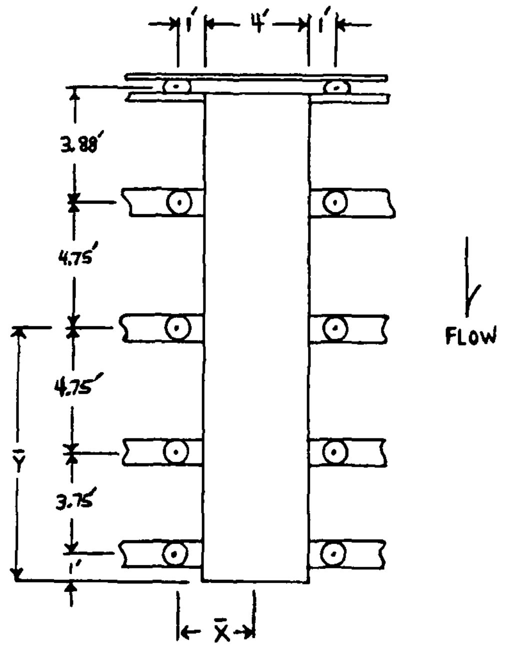

Figure 36. Moment of inertia of a pile group, Sandy Lake Dam

LOCATION OF CENTROID OF PILE GROUP

$\bar{Y} = \dfrac{(2)[1 + 4.75 + 9.5 + 14.25]}{8}$

$\bar{Y} = 7.38$ ft

$\bar{X} = (1/2)(6) = 3$ ft

MOMENT OF INTERIA OF PILE GROUP ABOUT CENTROID OF PILE GROUP

$I = Nd^2$

$I_{xx} = (2)[(7.38 - 1)^2 + (7.38 - 4.75)^2$
$+ (7.38 - 9.50)^2 + (7.38 - 14.25)^2]$

$I_{xx} = 199\ ft^4$

$I_{yy} = (8)(3)^2$

$I_{yy} = 72\ ft^4$

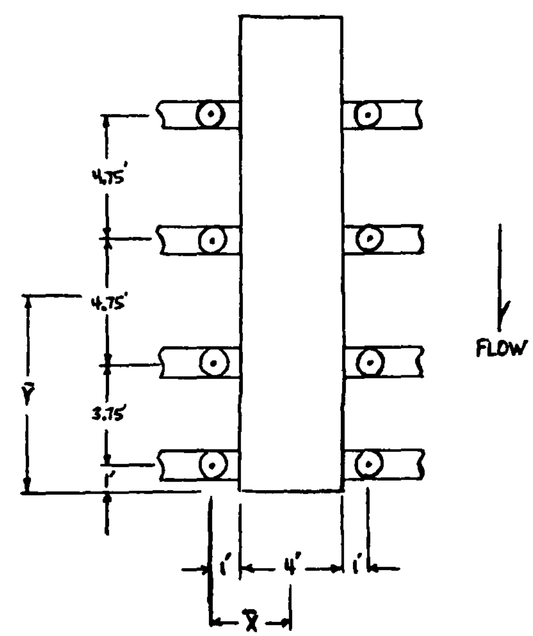

Figure 37. Moment of inertia of pile group, 8 piles, Sandy Lake Dam

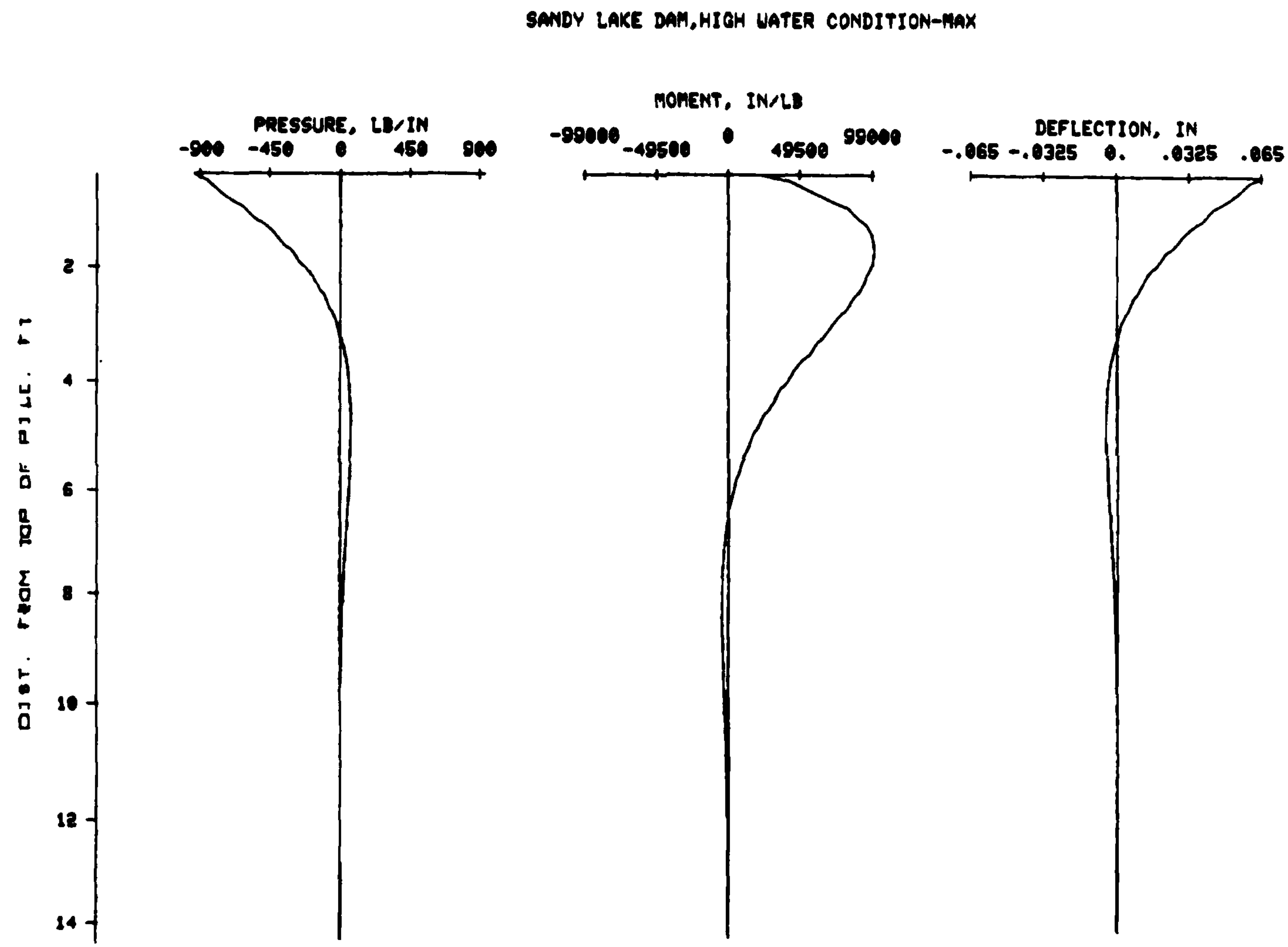

Figure 38. Pressure, moment, and deflection along the length of the most critically loaded pile

Table 1

General Reservoir Data

Location in miles above Ohio River	1106.85
Located on river	Sandy
Drainage area (square miles)	421
Original Operating Limits	
Stage	0.6 to 11.0 ft
Storage in 1000 acre-feet	79.4
Present Operating Limits	
Stage	7.0 to 11.0 ft
Storage in 1000 acre-feet	38
Ordinary Operating Limits	
Stage	7.0 to 11.0 ft
Storage in 1000 acre-feet	38
Flowate rights to stage	15+ ft
Maximum stage of record (1950)	17.51
Number of times upper operating limit exceeded	18
Number of times flowage limit exceeded	1
Maximum stage in 1950	17.51
Maximum discharge of record	3738
and year	1897
Elevation of gage zero: U.S.E. datum	1209.00
Elevation of gage zero: m.s.l. (1929 adj.)	1207.31
Year of first operation	1895
Normal spring stage drawdown	7.0 ft
Normal summer range	8.75 to 9.25
Desirable bridge clearance, 9.0 ft above reservoir stage of	11.0 ft

Table 2

Pertinent Dam Data

Control Structure	
Substructure	Round timber bearing piles with timber sheet-pile cut-off wall.
Superstructure	Gated multi-bay mass concrete sluiceways with concrete apron.
No. of sluiceways	12
Net width of sluiceways	66 ft
Pier size	14 ft high by 4 ft wide
Old lock size	211.5 ft long by 30 ft wide
Sluiceways	
No. of gated bays (includes fishway)	6
Size of steel slide gates	50 in. high by 60 in. wide
Width of bays	5.0 ft
Width of log sluice with stoplogs	11.0 ft
Elevation of log sluice and gate sills	1207.31
No. of bays in lock chamber (curtain wall with stoplogs)	5
Gross width of bays in lock chamber	6.0 ft
Lock floor elevation	1206.31
Sill elevation (top of curtain wall)	1216.81
Elevation of top of stoplogs	1219.31
Apron	
Type	Reinforced concrete on round timber piling with steel sheet-pile cutoff walls.
Length at lock chamber	200.25 ft
Length at gated sluiceways	101.75 ft
Total width (includes lock chamber)	109 ft
Apron protection	12-in. riprap with 12-in. filter 20 ft wide
Embankment	
Type	Earth fill with timber diaphragm
Top elevation left emb.	1227.81
Top elevation right emb.	1225.31
Length left	30 ft
Length right	75 ft
Approx. length of sandbag closure required (left and right)	150 ft

(Continued)

Table 2 (Concluded)

	Dikes			
Number	1	2	3	4
Location	Main Access road	Tieback to right emb.	Access road to camping area	Aitkin Lake Road
Length (ft)	1386	125	913	600±
Top elevation	1227.81	1225.31	1225.31	1226.81
Top width (ft)	20	10	11 to 20	20
Surfacing	Bituminous	Grass	Bituminous	Gravel
Slopes	1 on 3	1 on 3	1 on 1-1/2 backside	1 on 3
Riprap	680 lin ft	No	450 lin ft	No

Table 3

Locations of Pressuremeter Probe

Below the Bottom of the Piers

Hole	Test	Probe Location, ft
S-P1	Test 1	4.37
	Test 2	8.19
	Test 3	12.37
S-P5	Test 1	7.99
	Test 2	11.99
	Test 3	15.99

Table 4

Split-Spoon Test Results - Sandy Lake Dam

Hole	Test	Depth Below Bottom of Pier, ft	Number of Blows
S-P1	1	0.1- 0.6	4
		0.6- 1.1	4
		1.1- 1.6	1
	2	5.4- 5.9	6
		5.9- 6.4	6
		6.4- 6.9	11
	3	9.4- 9.9	8
		9.9-10.4	11
		10.4-10.9	13
	4	13.4-13.9	3
		13.9-14.3	1
		14.3-14.8	13
S-P5	1	0.7- 1.2	6
		1.2- 1.7	7
		1.7- 2.2	10
	2	9.5-10.0	8
		10.0-10.5	13
		10.5-11.0	15
	3	13.3-13.8	9
		13.8-14.3	10
		14.3-14.8	12
	4	17.3-17.8	7
		17.8-18.3	7
		18.3-18.8	9

Table 5

Unconfined Compressive Concrete Strengths

Core Hole	Specimen	Strength, psi
S-P1	S-P1M	5600
	S-P1B	6900
S-P5	S-P5T	7400
	S-P5M	5400
	S-P5B	3400
	Average Value ~ 5700	

Table 6

Patching Material for Cracks, Spalled Joints, and Holes

Material	Parts by Mass
Cement	100
Water	≈18 (adjust as needed)
Acrylic Polymer	27
Fine Sand (Passing 600-μm (No. 30) Sieve)	150

Table 7

Overlay Material for Surface Concrete Rehabilitation

Material	Parts by Mass
Cement	100
Water	≈20 (adjust as needed)
Acrylic Polymer	30

Table 8

Summary of Pile Foundation Loads, 10 Piles, Conventional Stability Analysis, Sandy Lake Dam

Case Loading	Horizontal Load F_H, kips	Number of Piles	Horizontal Load Per Pile, kips	F_V, kips	e (From Moment Center) ft	Moment about Center of Gravity of Pile Group $F_V(9.53 - e)$, kip-ft	Moment of Inertia of Pile Group, ft^4	Maximum Compressive Force Per Pile, kips	Maximum Tensile Force Per Pile, kips
Normal operation	46	10	4.6	205	7.11	496	385	31.5	None
Normal operation with truck loading (H15-44)	46	10	4.6	229	7.10	556	385	35.2	None
Normal operation with earthquake	53	10	5.3	205	6.81	558	385	32.9	None
Normal operation with ice	91	10	9.1	204.5	4.41	1047	385	43.7	-3.0
High-water condition	100	10	10	173	3.86	981	385	39.0	-4.6

Table 9

Summary of Pile Foundation Loads, 8 Piles, Conventional Stability Analysis, Sandy Lake Dam

Case Loading	Horizontal Load F_H, kips	Number of Piles	Horizontal Load Per Pile, kips	F_V, kips	e (From Moment Center) ft	Moment about Center of Gravity of Pile Group $F_V(7.38 - e)$, kip-ft	Moment of Inertia of Pile Group, ft^4	Maximum Compressive Force Per Pile, kips	Maximum Tensile Force Per Pile, kips
Normal operation	46	8	5.75	205	7.11	55	199	27.4	None
Normal operation with truck loading (H15-44)	46	8	5.75	229	7.10	64	199	30.7	None
Normal operation with earthquake	53	8	6.63	205	6.81	117	199	29.4	None
Normal operation with ice	91	8	11.38	204.5	4.41	607	199	45.0	None
High-water condition	100	8	12.5	173	3.86	668	199	43.0	-1.5

Table 10

Results of Direct Stiffness Analysis, 10 Piles, Sandy Lake Dam

Load Case	Elevation of Headwater, ft	Elevation of Tailwater, ft	Lateral Force at Top of Pile, F1 kips	Lateral Deflection At Top of Piles, U1 in.	Maximum Axial Force At Top of Pile F3, kips	Maximum Axial Deflection At Top of Pile, U3, in.	Minimum Axial Force At Top of Pile F3, kips	Minimum Axial Deflection At Top of Pile U3, in.
Normal operation	1220	1208	4.60	0.0280	31.43	-0.0385	9.37	0.0115
Normal operation with truck loading (H15-44)	1220	1208	4.60	0.0280	35.17	0.0431	10.42	0.0128
Normal operation with earthquake	1220	1208	5.28	0.0321	32.79	0.0402	8.00	0.0098
Normal operation with ice	1220	1208	9.10	0.0554	43.70	0.0535	-3.01	-0.0037
High-water condition	1227	1215	10.0	0.0609	39.08	0.0479	-4.68	-0.0057
Normal operation	1220	1212	4.22	0.0257	29.13	0.0357	8.96	0.0110
Normal operation with truck loading (H15-44)	1220	1212	4.22	0.0257	32.84	0.0402	10.04	0.0123
Normal operation with earthquake	1220	1212	4.90	0.0298	30.49	0.0374	7.59	0.0093
Normal operation with ice	1220	1212	8.72	0.0531	41.39	0.0507	-3.41	-0.0042
High-water condition	1227	1220	8.26	0.0503	34.38	0.0421	-3.35	-0.0041

Table 11

Results of Direct Stiffness Analysis, 8 Piles, Sandy Lake Dam

Load Case	Elevation of Headwater, ft	Elevation of Tailwater, ft	Lateral Force at Top of Pile, F1 kips	Lateral Deflection At Top of Piles, U1 in.	Maximum		Minimum	
					Axial Force At Top of Pile F3, kips	Axial Deflection At Top of Pile, U3, in.	Axial Force At Top of Pile F3, kips	Axial Deflection At Top of Pile U3, in.
Normal operation	1220	1208	5.75	0.0324	27.30	0.0335	23.69	0.0290
Normal operation with truck loading (H15-44)	1220	1208	5.75	0.0324	30.58	0.0375	26.39	0.0323
Normal operation with earthquake	1220	1208	6.59	0.0372	29.27	0.0359	21.56	0.0264
Normal operation with ice	1220	1208	11.37	0.0641	45.03	0.0552	4.58	0.0056
High-water condition	1227	1215	12.50	0.0705	41.14	0.0504	0.58	0.0007
Normal operation	1220	1212	5.27	0.0297	25.18	0.0309	22.45	0.0275
Normal operation with truck loading (H15-44)	1220	1212	5.27	0.0297	28.42	0.0348	25.19	0.0309
Normal operation with earthquake	1220	1212	6.12	0.0345	27.14	0.0333	20.33	0.0249
Normal operation with ice	1220	1212	10.90	0.0614	42.89	0.0526	3.35	0.0041
High-water condition	1227	1220	10.33	0.0582	35.86	0.0439	1.87	0.0023

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